Analysis of a Three-Span Continuous Grillage Having Varying Section Properties

by

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1. Introduction

Simply-supported bridge-decks are frequently designed as grillages so that the interaction between longitudinal main members can assist the load distribution when the deck has to carry the abnormal heavy vehicle (the H.B. loading of B.S.153). The analysis of simply-supported single-span grillages is now well established by the use of the 'quasi' slab technique.[1] In recent years the number of continuous or otherwise statically-indeterminate bridge decks—particularly in prestressed concrete—has been increasing and where continuous decks have been adopted they have often been constructed with beams of varying depth and section properties. A demand has, therefore, arisen for an analytical technique for such continuous grillages.

For some time now the quasi-slab method for simply-supported single span grillages has also been applied for continuous grillages by the use of certain simplifying assumptions as mentioned later in this paper. The results of this method were initially tested by using Rowe's treatment for indeterminate grillages,[2] but due to varying section properties and the fact that the spans were not 'fixed' or 'semi-fixed', some broad simplification was required even in that treatment. However, the use of the 'equivalent simply-supported span' technique as described below gave results which were satisfactory to a number of designers and in one case known to the authors[3] these results were checked by a model and found to be satisfactory.

In the recent design of a proposed bridge over the River Trent at Newark, the same 'equivalent simply-supported span' technique was adopted for the analysis of load distribution of the three-span continuous bridge deck having varying section properties, the deck being designed as a grillage for abnormal load. After this analysis had been carried out two of the authors fortuitously discussed a general computer programme developed at Leeds University which could provide a detailed and rigorous analysis of such a brick deck. It was decided that the programme should be tried to obtain a rigorous analysis of the Newark bridge deck for midspan design criteria since this would afford a very useful check on the 'equivalent simply-supported span method' as adopted in this instance and elsewhere as a general method for this type of structure.

2. Design

Fig. 1 indicates the type of bridge deck under discussion. It consists of nine longitudinal prestressed concrete box beams continuous over three spans of 130 ft., 260 ft. and 130 ft. respectively, and having varying moments of inertia. These are joined transversely by the deck slab and by diaphragms at 16 ft. 3 in. centres, the depth of the diaphragms varying in accordance with the depth of the longitudinal beams. The live loading adopted for this bridge was :—

(a) The Ministry of Transport normal live load (equivalent loading curve) or—

(b) A combination of the abnormal H.B. loading (B.S.153) together with the whole or a proportion of normal loading on other carriageways. The bridge provides dual carriageways of 24 ft. width together with 8 ft. flush hard shoulders and 1 ft. marginal strips at each side and the position of the abnormal load vehicle can be anywhere on the whole 34 ft. width on each side.

For analysis under normal live loading, the elastic properties of each separate beam were evaluated by means of a digital electronic computer—an 'autocode' programme being developed for this purpose jointly with Professor P. B. Morice of Southampton University. This programme required the introduction only of the beam dimensions at a number of sections in each span and yielded the influence lines for reactions and for bending moments for these sections. This enabled the analysis of the three-span continuous beam to be carried out extremely quickly for normal live load.

The intention in the design, i.e., the proportioning of beams, diaphragms, etc., was to ensure that the maximum value of bending moments in any beam under the abnormal live load case with transverse distribution taken into account, were about the same as the maximum values of bending moment in a single beam under normal live load. If this could be achieved and tested for accuracy then, apart from one or two detailed considerations of shear, the abnormal live load case would not govern the design of the main longitudinal members.

3. The 'Equivalent Simply-supported Span' Technique

For the original analysis of the bridge deck as a grillage, this simplified method was used. The abnormal vehicle and combination of normal live load

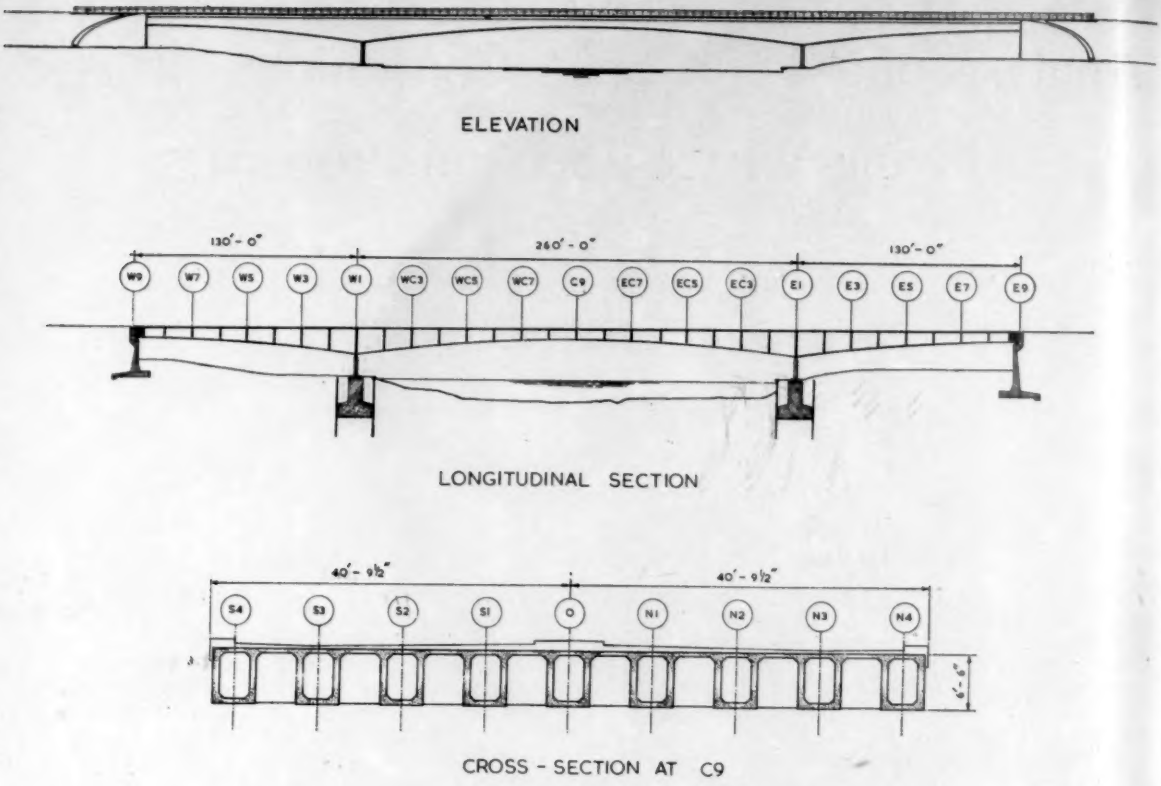
ELEVATION
130'- 0"
260'- 0"
130'- 0"
W9 W7 W5 W3 W1 WC3 WC5 WC7 C9 EC7 EC5 EC3 E1 E3 E5 E7 E9
LONGITUDINAL SECTION
40'- 9½"
40'- 9½"
S4 S3 S2 S1 O N1 N2 N3 N4
CROSS - SECTION AT C9

Fig. 1.—Main Features of the particular Bridge considered, the Winthorpe Bridge on the Newark By-Pass

was first positioned near the middle of the central span and all of it was assumed to be acting on one beam. The precise position was chosen to give the maximum longitudinal live load bending moment at the centre. By using the influence lines previously referred to, the positions of the points of contraflexure in the central span of the beam were readily determined; in this instance, these points were 122 ft. apart. This length was then taken as the equivalent statically-determinate simply-supported span. It will be seen from Fig. 1 that the variation of section properties is not very large over this length since the depth does not change very much near the middle of the central span. An equivalent bridge deck was then considered having a width the same as the actual bridge, a 122 ft. simply-supported span and a constant longitudinal moment of inertia (an average value to accommodate the small variations). For such a bridge deck the standard methods of grillage analysis can be readily used. The analysis was carried out in a form to yield distribution factors which are the factors by which the average longitudinal bending moment (i.e., total moment divided by the number of beams) has to be multiplied in order to obtain the moment occurring in each beam. Transverse influence lines were used for the distribution factors so that the heavy vehicle and the appropriate portion of normal live loading could be positioned transversely to produce the maximum possible longitudinal bending moment. Generally this occurs in the edge beam with the heavy vehicle as near as it can get to the edge of the bridge deck.

A similar procedure was carried out for transverse bending moments in the diaphragms, the critical position of the heavy vehicle for maximum sagging bending moment occurring when the vehicle is as near to the centre of the deck as possible. Thus, both the longitudinal and transverse bending moments were obtained by the standard routine grillage analysis. Unfortunately, the coefficients and tables which have been prepared for the quasi-slab method apply only for bending moments, but the maximum values of shear can be obtained by plotting the various bending moment diagrams and measuring the slopes of the curves. This gives a reasonable assessment of shear but is somewhat sensitive to errors when the curves are drawn through a limited number of plotted points; for accuracy a great deal of work is required.

From this method of analysis it was established that the intention of having similar values of maximum bending moments for normal live load and abnormal live load had, in fact, been achieved. For the analysis of sections of the bridge deck other than at the middle of the central span, the same distribution factors for each beam were used. This is considered to be on the safe side (confirmed by results) and as the normal live loadings provide the criteria for design a more rigorous analysis of the side spans is not required.

4. Computer Analysis

A master programme for the analysis of grid frameworks, using the generalised slope-deflexion method, was developed in the Department of Civil Engineering at the University of Leeds, for the Ferranti Pegasus computer during the autumn of 1958. This first programme can solve any grid framework composed of prismatic members rigidly-jointed at the interior

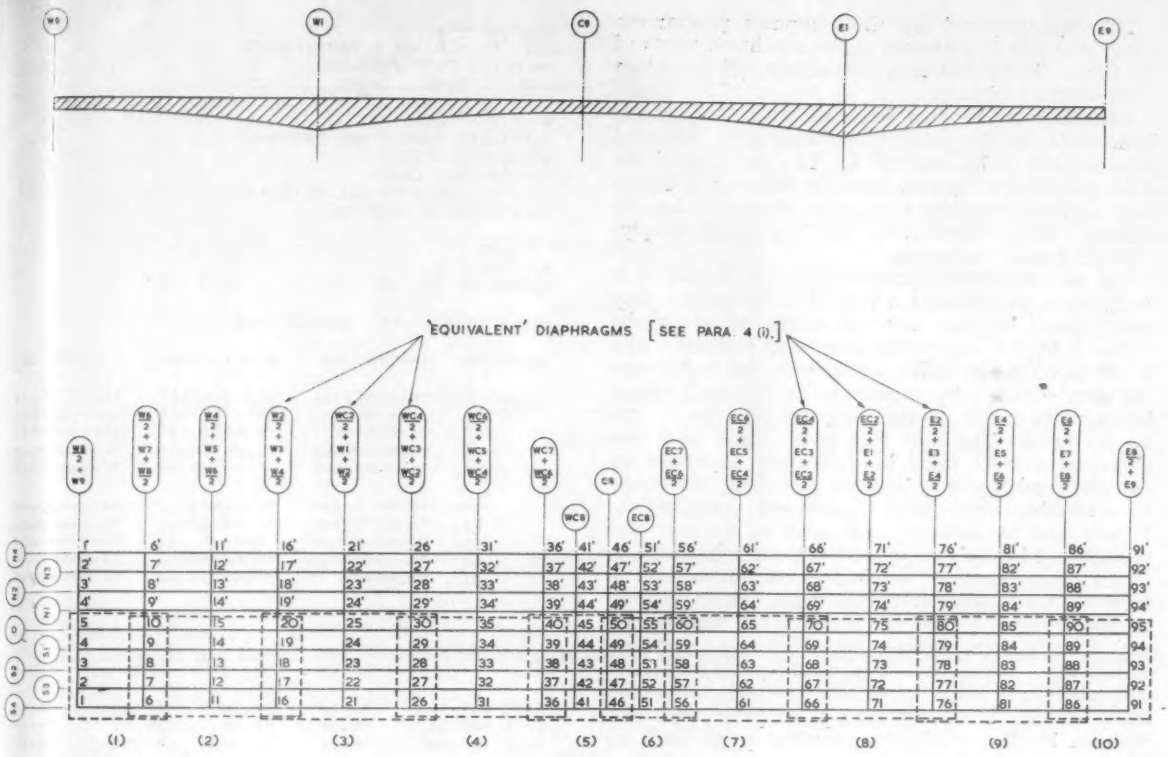

Fig. 2.—Grillage as analysed

joints, orientated and connected in any prescribed manner, provided there are no more than eighteen joints and fifty-six members. The grid may be supported at the joints in any prescribed manner by lateral and rotational restraints up to forty in number and any specified loading may be applied. Only the basic data describing the structure and the loadings have to be inserted on the data tape and the results of the analysis are printed out by the computer in a self-explanatory table.

The first programme was quickly supplemented by a partitioned grid framework programme which analyses any gridwork provided it can be divided into a number of linearly-connected sub-frames, each with less than eighteen joints. For convenience in use this programme requires the additional storage facility of magnetic tape, but there is no complication in the data preparation and in the print-out of results. Both these programmes have already been reported [4,5,6] and their use demonstrated in a number of different applications. The analysis of the three-span bridge grillage described here is, however, the most complicated of any yet attempted.

The method of preparing the data for the computer solution of this particular bridge is different from that for the quasi-slab analysis. The latter assumes uniformly-distributed stiffness in the longitudinal and transverse directions, whereas the former solves an open interconnected system of main beams and transverse diaphragms with the flexural and torsional stiffnesses assumed concentrated at the centre lines of the members.

The actual bridge consists of nine identical main beams of varying section, spaced at 9 ft. 6 in. centres, with diaphragms at 16 ft. 3 in. intervals. The number of points of intersection of the main beams with the transverse diaphragms is very large, and although the computer programme could cope with a structure of this complexity, it was decided that the number of joints should be reduced for convenience. A simplification was accomplished in two ways :—

(i) by omitting alternate diaphragms, except for the two adjacent to the central diaphragm, and by adding half of each omitted diaphragm to its immediate neighbours, the number of points was reduced to 171.

(ii) by analysing only half of the bridge for the cases of symmetrical and anti-symmetrical loadings and later superimposing the results to obtain the complete solution the number of nodes was thus reduced to 95 as shown in Fig. 2.

The computer programme can, at present, solve grid frameworks consisting only of straight uniform members. It was therefore necessary to replace each longitudinal beam of varying section by several shorter members each with constant sectional properties. Thus the segments of members between the nodal points were taken to be uniform.

This equivalent grid framework was divided into ten sub-frames, two with ten and eight with fifteen nodes each, as shown in Fig. 2. The computer programme can actually handle sub-frames with eighteen joints, but for simplicity in data preparation and checking it was decided to keep the sub-frame division 'symmetrical' with respect to the transverse centreline of the bridge and parallel to it.

The data required by the computer programme consists of the co-ordinates of the numbered joints of the frame, the properties of the sections and the values of the applied loadings.

For maximum bending moments in the longitudinal beams and in the transverse diaphragms two load criteria have to be considered. The positions of the abnormal vehicle together with the Ministry of Transport equivalent normal loads were obtained from the influence lines evaluated by the 'equivalent simply-supported span' technique.

The grid frameworks programme solves frames most easily when point loads are applied at the joints. Any load applied between any two nodes can always be replaced by its equivalent fixed-end moments and forces at the nodes. The actual moments and shears are then obtained by superimposing the local effects between the nodes on the computer solution for the whole frame. The fact that point loads from the abnormal vehicle could be positioned anywhere on the rectangular panels between the main and transverse members gave rise to an additional complication. It was realised, however, that most of the load was transmitted by slab action to the nearest beams, i.e., to the main beams of the bridge. Since the slab is very flexible compared to the main beams, it was therefore assumed that interpanel loads were transmitted to the main beams as their statically-equivalent *point* loads. These were then again split up as statically-equivalent loads at the adjacent nodes along the main beams. Thus any inter-panel point load was replaced by four statically-equivalent point loads at the corners of the panel.

For the transverse criterion the wheels on one axle of the abnormal vehicle were positioned over the central diaphragm. The fixed-end moments at the nodes on this diaphragm were therefore included. For a theoretically-accurate solution the corresponding fixed-end forces should have been taken, but the end forces were inadvertently taken as though the diaphragm members were simply-supported at the nodes. Since many approximations had already been incorporated in the analysis, such as treating the box beams with two webs as though they were plate girders with single webs at the centre line of the main beams, it was decided that these statically-equivalent forces, which are little different from the fixed-end forces in any case, were accurate enough for this analysis.

Since only one half of the bridge, up to and including the longitudinal centre-line of the bridge, was to be analysed by the programme, the loading for the symmetrical and anti-symmetrical cases had to be proportioned in such a way that the superpositioning of the results of the two analyses would give the correct solution for the whole frame. The usual $+\frac{W}{2}+\frac{W,}{2}$ and $+\frac{W,}{2}-\frac{W}{2}$ superpositioning technique was therefore utilized. For the symmetrical case zero rotations were imposed along the longitudinal centre line, and zero vertical displacements for the anti-symmetrical case.

The actual preparatory work for the computer analyses was almost identical for the two cases, the data tapes being different only in the loading and support conditions. As mentioned earlier, the bridge was divided into ten sub-frames, connected linearly. The computer solves these in succession, storing all the intermediate results and sub frame data on magnetic tape operating alongside the computer. After

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8/5/59----43
GRID FRAMEWORKS - PARTITIONED
MAGNETIC TAPE VERSION
9 / 3 / 1959

8/5/59----44
R.TRAVERS MORGAN AND PARTNERS
WINTHORPE BRIDGE
DESIGN LIVE LOAD
CASE A. LONGITUDINAL CRITERION
(SYMMETRICAL LOADING)


8/5/59----45
SUB-FRAME NO. 10

DEFLECTIONS   FT. AND RADIANS

JOINT NO.   X-ROTATION      Y-ROTATION      VERTICAL

      86   -0.00000251     -0.00129961     +0.05906704
      87   -0.00000225     -0.00130066     +0.05908980
      88   -0.00000171     -0.00130156     +0.05910871
      89   -0.00000098     -0.00130218     +0.05912123
      90   +0.00000000     -0.00130240     +0.05912557

      91   -0.00000005     -0.00207640     +0.00000000
      92   -0.00000001     -0.00207690     +0.00000000
      93   -0.00000002     -0.00207732     +0.00000000
      94   -0.00000001     -0.00207759     +0.00000000
      95   +0.00000000     -0.00207770     +0.00000000

MEMBER        FORCES TONS.        MOMENTS TON.FT
NO.           SHEAR   TORSION     END 1     END 2

158( 91> 92)  +0.00   +0.00       +0.03     +0.01
159( 92> 93)  +0.00   +0.00       +0.01     +0.01
160( 93> 94)  +0.00   +0.00       +0.01     +0.01

161( 94> 95)  +0.00   +0.00       +0.00     +0.00
162( 86> 91)  +2.03   -0.02      +65.98     -0.01
163( 87> 92)  +2.03   -0.02      +65.93     -0.00
164( 88> 93)  +2.03   -0.02      +65.89     -0.00
165( 89> 94)  +2.03   -0.01      +65.86     -0.00

166( 90> 95)  +1.01   +0.00      +32.93     -0.00
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Fig. 3.—Typical Print-out of Computer Results

solving the last sub-frame the computer holds the complete matrices of the coefficients of all the simultaneous equations required for the analysis of the entire frame; the last deflexion is then eventually evaluated, and thence all the others by back substitution. The print-out for each sub-frame consists of the vertical deflexion and two rotations about mutually perpendicular axes at each joint; also the shear and torque in each member and moments at each of the two ends thereof. A typical print-out is shown in Fig. 3. Ten such sets gave all the results for one case of loading on half the frame. The time taken for one such solution was 45 minutes on the computer. In all $95 \times 3 = 285$ simultaneous equations were solved, in each analysis. (This number was effectively reduced by 39 when the imposed zero restraints were allowed for.) The computer time for one set of loading (allowing for super-positioning) amounted to 1½ hrs.; three hours machine time was required for the two design criteria. The time taken for the preparation of data for the computer (for calculating the nodal forces and fixed-end moments, preparing the data tapes with checking at every stage), amounted to about 30 man hours for the two design criteria.

A large amount of work was performed on the output of results, since all the forces and moments were superimposed to obtain the final results. In this particular analysis the values at every joint were

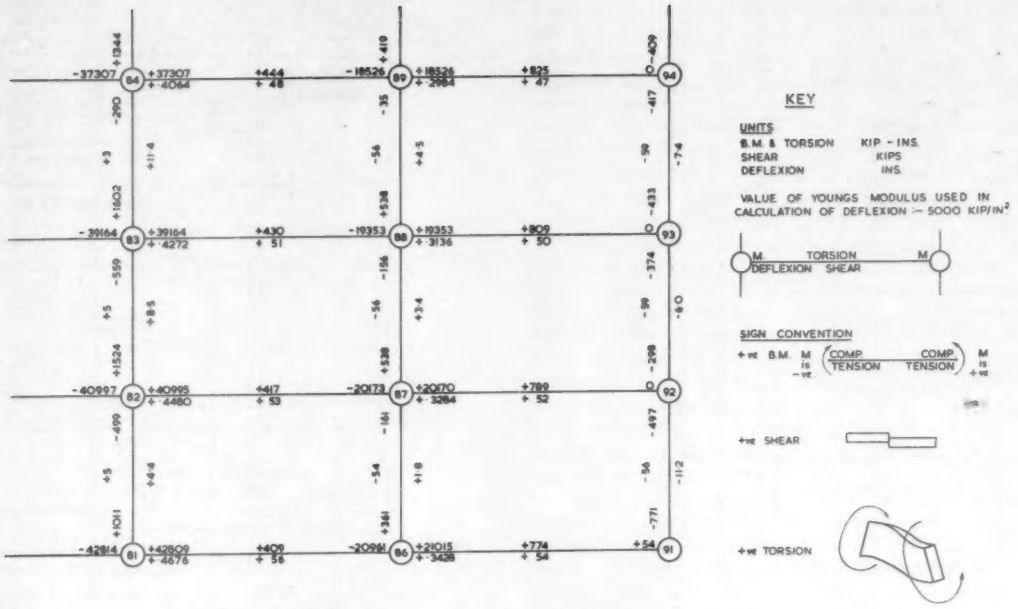


Fig. 4.—Some of the Forces and Moments obtained by super-positioning for the Longitudinal Loading Criterion.

found in this way. They were not all actually needed for design purposes—the maximum values are of most interest in these loading cases. A check on statical and deformational compatibility at joints was successfully applied to ensure that the structure had been solved correctly. The results could then all be used with confidence.

Influence surfaces could have been obtained, if required, by positioning unit loads at the various nodes shown in Fig. 2 and by plotting the results to give the various required influence surfaces. The complete analysis, to be adequate, would involve the solution of 21 loading cases, which include twelve analyses for the symmetrical and anti-symmetrical cases for the whole bridge, eight analyses for half the bridge and one for a quarter of the bridge, (because of the axes of symmetry). A total of about $(90 \times 12 + 45 \times 8 + 22\frac{1}{2} \times 1)$ minutes, i.e. $24\frac{1}{2}$ hours of computer time would therefore be required.* The output of results would not involve a great deal of time and labour; once the first data tapes were prepared correctly only the loads need be changed in subsequent analyses. Also fixed-end forces and moments would not need to be calculated since point loads would be applied at the joints; this would simplify the preparation of data tapes considerably. The results thus obtained would enable the designer to position the loading quite accurately, anywhere on the bridge, to obtain the worst loading conditions for any particular member but would require a considerable amount of 'post-computer' numerical work.

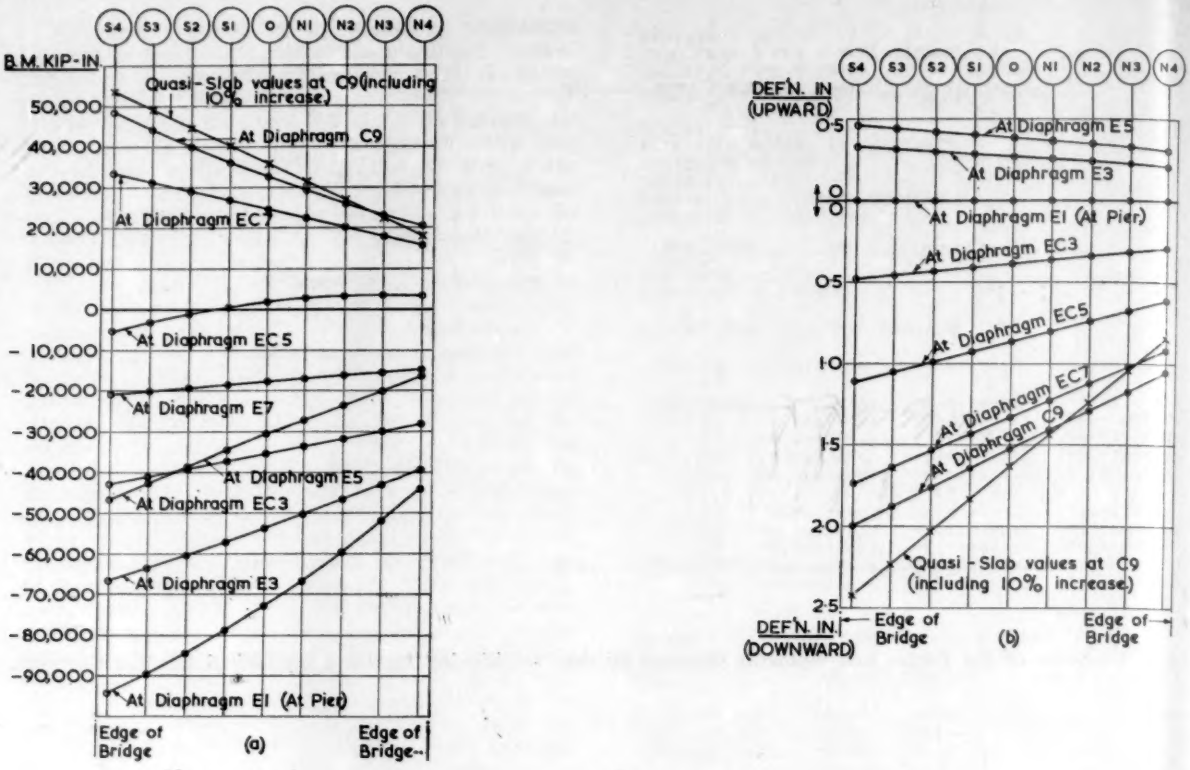
*The 45 and $22\frac{1}{2}$ minutes are over-estimates. Also, the total time required would be reduced by about one-third with the "alternative loading facility" which has now been built into the programme. Use of this facility simplifies the preparation of data tapes appreciably in an application like this.

The loading positions for the worst longitudinal and transverse bending moments were determined from the approximate analysis described in the previous sections and were assumed to be correct. The computer solution then provided a check on the accuracy of the approximate analysis for the two load criteria; the results of these two analytical methods are compared in section 6 below.

5. Work of Output

A considerable amount of drawing office work was performed on the output data, but this labour could be considerably reduced if only the values affecting the design were considered. For each load criterion the symmetrical and anti-symmetrical results of the moments, forces and deflexions had to be superimposed and added to give the final values. This was done on a diagrammatic plan grid and Fig. 4 shows a section of this plan grid with the resulting values. Some care was necessary when inserting values for the half of the bridge not covered in the computer grid to ensure accuracy of signs. A check of this post-output work was obtained by applying the simple statical rule of summation of bending moments and torsions to check the equilibrium at each node. Additional checks for vertical forces and (of less significance) for displacements are also possible. Discrepancies discovered in this way were entirely due to human error in the post-output work, the actual output being found to be entirely self-checking.

It will be appreciated that much of the data obtained in this way was in excess of that normally required for design, but much useful information was obtained which was not otherwise available by previous methods.



(a) Longitudinal Bending Moments in Beams at Diaphragms.
(b) Deflexions of Beams at Diaphragms.

NOTE :- All values are plotted from Computer results unless marked otherwise.

Fig. 5.—Longitudinal Criterion.

For example, torsional moments are not normally obtained by the quasi-slab method. Since only two load positions were analysed, only the bending moment criterion values at the middle of the central span were obtained, many of the other values, e.g. at the supports, not being the maximum to be allowed for. This same limitation, of course, applies to other methods when only two loading positions are considered.

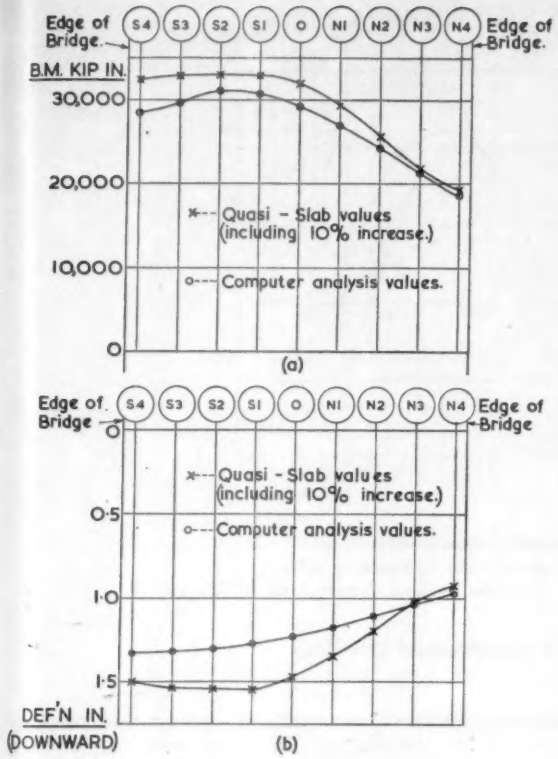
From the completed grid plans mentioned above various graphs could be drawn comparing the computer results with those obtained from the approximate quasi-slab analysis. In plotting any bending moment or shear values allowance had to be made for the 'fixed-end moments' or 'fixed-end shears' used in the computer input, and the inter-joint values had to be superimposed, as explained earlier.

6. Comparison of Results

Fig. 5(a) shows the longitudinal bending moment in the beams plotted transversely for various longitudinal stations. The loading in this case was that leading to the longitudinal bending moment criterion. The moments obtained by the quasi-slab analysis at the centre of mid-span are also plotted and can be compared with the moments by computer result at mid-span. Fig. 5(b) is a similar comparison for deflexions instead of moments. It can be seen that the comparison of bending moments between the computer analysis and the quasi-slab 'equivalent simply-supported span' analysis for mid-span is very good, the latter being ten per cent greater at the maximum values. The quasi-slab analysis values include an approximate ten per cent allowance to account for the fact that only the first load term of the longitudinal load series is used. If this ten per cent had not been added, the maximum values would have been almost identical. By comparing the deflexions for this load case it can be seen that fair agreement exists though the difference in maximum deflexion is about 23 per cent with the quasi-slab results higher. The transverse deflexion profiles obtained from the computer analysis are approximately the same shape as is assumed by the quasi-slab method.

Fig. 6(a) shows the longitudinal bending moments at the central diaphragm transversely as in Fig. 5(a) but for the transverse load criterion. This comparison was made for interest only since this load criterion does not give anything like maximum longitudinal bending moments. Fig. 6(b) shows the deflexion comparison as in Fig. 5(b) but for the transverse load criterion. The agreement both for moments and deflexions is very good, the quasi-slab values being generally a little higher than the computer values.

With regard to the longitudinal bending moments at positions other than mid-central span, the quasi-slab analysis was carried out by using the distribution factors as mentioned in the foregoing. Taking, for example, the pier position, the quasi-slab results



(a) Longitudinal Bending Moments in Beams at Diaphragm C9.
(b) Deflexions of Beams at Diaphragm C9.

Fig. 6.—Transverse Criterion.

cannot be compared directly with the computer results, since the former analysis was carried out in full to obtain maximum bending moment values at all sections whereas the computer analysis was performed on no more than two special load cases, thus yielding maximum bending moment values at mid-central span only. However, a further calculation was made using the quasi-slab method to provide longitudinal bending moments at the pier for the load positions used in the computer work for the longitudinal criterion. It is of interest to know that whereas in the mid-central span comparison the computer results were about 91 per cent of the quasi-slab results, for the pier positions the computer results were about 85 per cent of the quasi-slab results. This confirms the technique whereby the distribution factors for mid-span are calculated and are then applied to the total maximum bending moments occurring at other sections, any small error being on the safe side. Fig. 7 shows the transverse bending moment in the mid-central span diaphragm for the transverse load criterion. The maximum value on this graph is in fact the design criterion. It can be seen that comparison between the computer results and the quasi-slab results is again quite good. The latter is generally higher than the former and for the maximum values the difference is about 20 per cent.

Fig. 8 shows the same comparison for the longitudinal load criterion. This is done for interest only since it can be seen that the transverse bending moment for this load position is relatively small. The general shape of the curve is similar in this instance but the agreement is not as good as in other cases. This is due in some part to the discontinuities which occur in the computer analysis at the nodes and is referred to under 'Conclusions.'

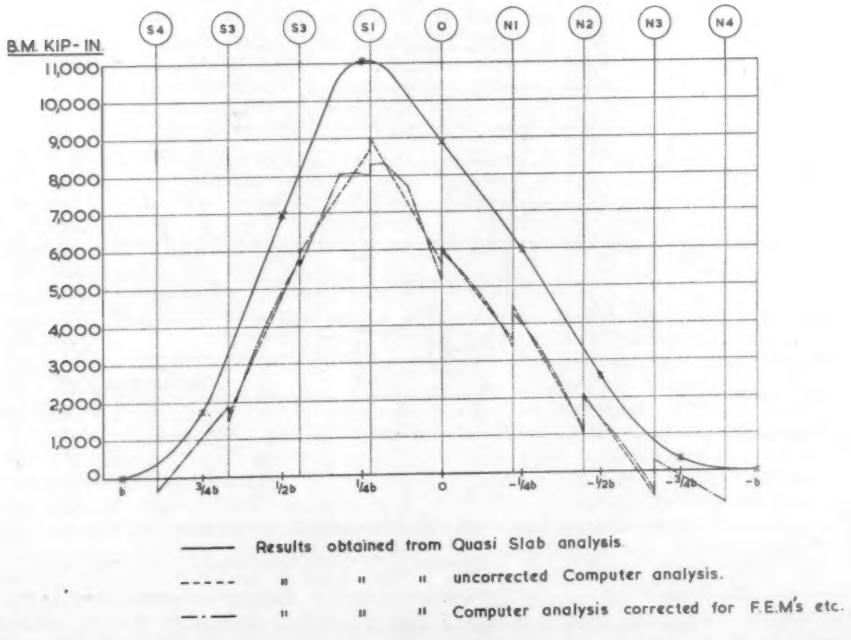


Fig. 7.—B.M. for Diaphragm C9 (Transverse Criterion).

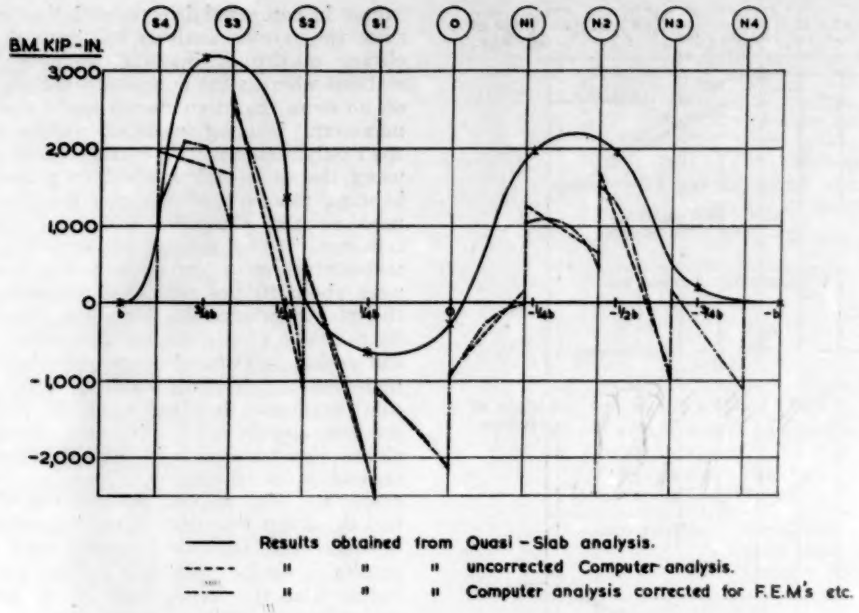


Fig. 8.—B.M. for Diaphragm C9 (Longitudinal Criterion)

Fig. 9 shows the shear force in the mid-central span diaphragm for the transverse load criterion. The agreement is reasonable though the angular shape of the curve from the computer values reflects the assumption of point loads and single webbed members and some rounding-off is required; equally the smooth curve from the quasi-slab analysis reflects the assumption of a uniform cross-section and a curve with local undulations would be nearer actuality. An attempt has been made to correct the computer results by introducing the 'free end' shears and considering the reactions at the beams (taken as single line beams in the computer analysis) as divided between the two webs. A further modification could be made to smooth out the projecting 'fingers' in the diagram by taking the load from each web as distributed over the web width instead of as a point load at the web centre. The limited number of stations

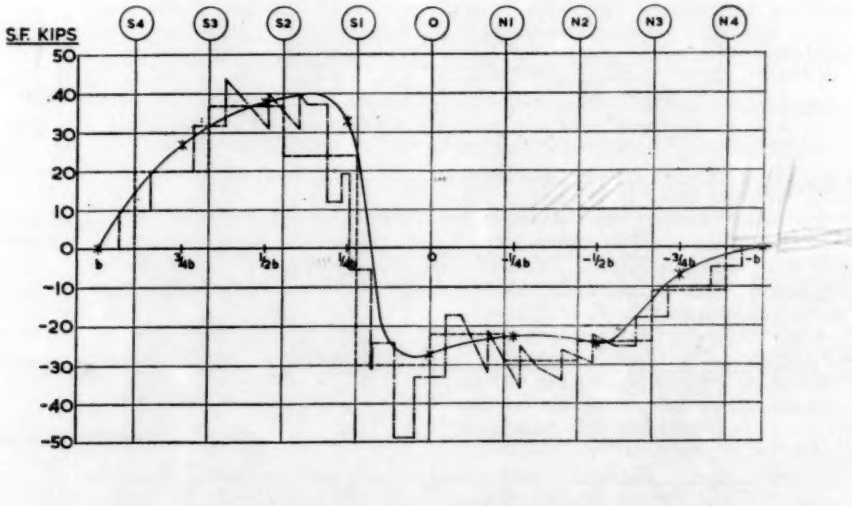


Fig. 9.—Shearing Force Diagram for Diaphragm C9 (Transverse Criterion).

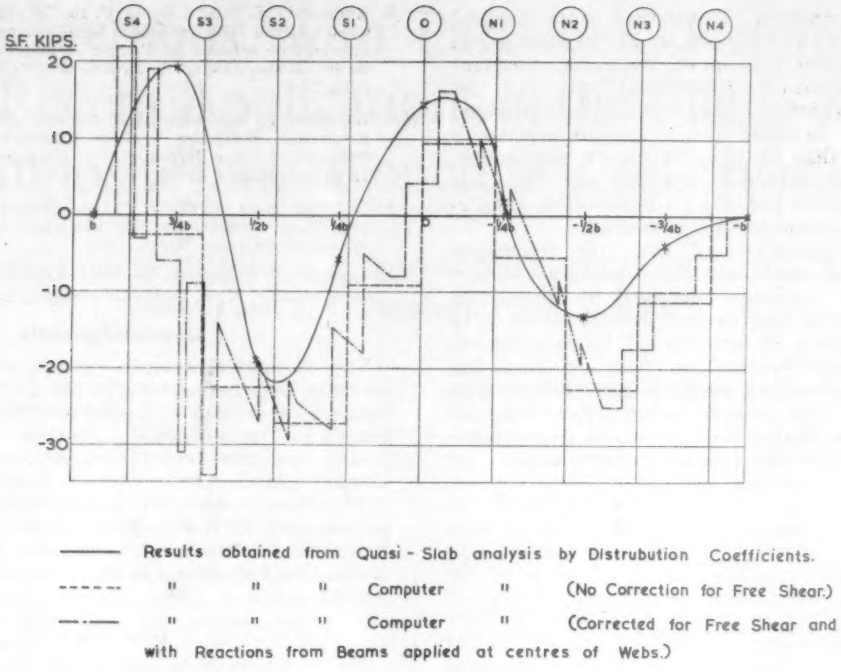


Fig. 10.—Shearing Force Diagram for Diaphragm C9 (Longitudinal Criterion).

in the quasi-slab shear analysis is also likely to have affected this agreement.

Fig. 10 gives a similar comparison of shear, but for the longitudinal criterion. Although maximum values are not too different in the two analyses, a comparison cannot well be made due to the aforementioned difficulties. It should be noted that the last two figures are given for interest only, since in neither case are the values of shear as great as the criterion values. This is due to the fact that the two load positions were devised to give bending moment criteria and in the absence of influence surfaces from the computer, as mentioned above, maximum values of shear in the diaphragms could not be ascertained from the computer results. The criterion values were obtained from the shear influence lines evaluated by the quasi-slab method, the maximum positive values of shear being nearly 70,000 lb. and the negative value nearly 60,000 lb.

7. Conclusions

It is apparent that the rigorous computer analysis confirms the 'equivalent simply-supported span' simplification made in the quasi-slab analysis in this instance. The comparison of results shows very good agreement in the case of longitudinal moments and transverse moments and reasonable agreement for shear forces. Since the bending moments of the quasi-slab technique are confirmed, it is reasonable to consider that the absolute maximum values of shear forces obtained in the quasi-slab analysis from the bending moment diagrams are also reasonably accurate.

There is a difficulty in relating the computer results directly to the quasi-slab results. This is because in the computer work the structure is considered as a pure grillage (though allowing for tee-beam action) and is thus broken down to a set of nodes. These nodes are reduced in number, if necessary, until they come within the convenient application of the programme. The results obtained from the computer, unless they are modified, tend to exaggerate sharp changes in the bending moments, the torsion and the shearing forces. In practice, these discontinuities would not occur in the abrupt manner shown on the graphs. The quasi-slab analysis which is based on substituting for the actual grillage an equivalent slab ignores local effects though they may actually occur. In this particular structure the use of box beams having two webs produced a structure rather unfavourable to a grid framework analysis. Nevertheless, in being able to analyse the representation of the entire bridge structure this computer method of analysis has afforded an invaluable check on the quasi-slab method for simply-supported spans in its extended application to three-span grillage type bridges. The agreement between the various values obtained from the two methods is most reassuring.

The quasi-slab method has considerable attractions for drawing office procedure, but care must be taken in adopting it for related kinds of structures. The grid framework programme, on the other hand, supplies useful additional information on shear forces, torque, and displacements. Designers may consider this an appreciable advantage in spite of the super-positioning and the rounding-off required over

the widths of members. The overall times involved for design by the two methods are of the same order. The cost of machine time is, of course, an important factor when charged on a commercial basis. It seems clear that when there is a large number of longitudinal and transverse members the quasi-slab method is easier to apply than the grid framework programme, but the computer method is preferable when there are not too many joints and when an equivalent simply-supported span cannot be safely assumed.

From this application of the grid framework programme it is seen that the 'equivalent simply-supported span' technique obtained by considering a simply-supported longitudinal member leads to a sufficiently accurate design method for symmetrical three-span grillage type bridges. This conclusion has since been checked for another such bridge with different spans and section sizes.[6] The computer method may be further applied to investigate other possible applications of the quasi-slab method.

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DISCUSSION

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by November 30th, 1961.

Book Reviews

Design of Steel Structures, by Boris Bresler and T. Y. Lin. (New York and London: Wiley, 1960). 9½ in. × 6¼ in., 710 plus xiii pp., 78s.

This book contains a vast amount of information relative to both the theory of structures and the practical applications of such theory, and this is presented in a manner which makes for easy reference.

A great deal of the information is of necessity of an elementary character but its collection into one volume does materially assist the student.

The chapters dealing with beams are particularly interesting as a great deal of attention is given to torsion—a subject difficult to find in so detailed a manner in most text books.

A chapter is also provided on light-gauge members which should prove of value having in mind the increasing use now being made of this type of section, and examples are also given of the problems involved in designing stiffened sheet wall panels etc.

Throughout the book the text is amply illustrated by clear sketches and graphs, and these are well supported by adequate calculated examples. References are given at the end of each chapter which will prove of great value to anyone seeking to collate information on a particular aspect of structural engineering.

Undoubtedly, this volume will form a useful addition to the bookshelf of both student and qualified engineer and is a fit companion to the "Steel Designers' Manual." W.B.

Design of Modern Steel Structures, 2nd Edition, by L. E. Grinter. (New York: Macmillan Company, 1960). 9 in. × 6 in., 491 plus xxi pp., 45s. 6d.

This valuable book forms a companion to the "Theory of Modern Steel Structures," Vol. 1 by the same author. Taken together they cover most of the problems of analysis and design included in the usual undergraduate course in statically determinate structures. The book is by no means limited to steelwork, as its title would imply, but also covers many of the problems which arise in heavy timber construction.

The format is the same as in the first edition, joints and connexions, riveted and welded and timber connectors, being dealt with first, followed by chapters on tension members, compression members, rolled beams and girders and combined axial and flexural stresses, and a chapter on maximum permissible stresses.

Separate chapters are devoted to the design of four important structures, a building girder and a roof truss, each designed both for welding and riveting, a low-truss highway bridge designed for riveting and a preliminary design for a multi-storey office building with welded joints. In general, of course, American specifications are used.

In place of the chapter on design of continuous beams, this second edition has a section on plastic design of statically indeterminate steel structures.

Model Methods, with Particular Reference to Three Recent Applications in the Fields of Steel, Composite and Concrete Construction*

Discussion on the Paper by Professor S. R. Sparkes, Ph.D., M.I.Struct.E., M.I.C.E. (Member of Council) and J. C. Chapman, Ph.D., A.M.I.C.E.

THE PRESIDENT, introducing the lecturers, said he had known Professor Sparkes very well for many years. After graduating at the University of Bristol in 1932, he had worked for Dorman Long & Co., Ltd., where he had learned much about structural steelwork. Col. Kirkland remembered him there, particularly for his most brilliant handling of a compound loading problem solved by calculus. He had shown Professor Sparkes an arithmetical solution ; since then he had been a most able collaborator and a very dear friend.

The Authors presented the paper, which was illustrated by many slides.

Professor SPARKES considered himself to have been fortunate to have had Colonel Kirkland as his Chief when he was with Dorman Long and Company. Amongst many other things, Colonel Kirkland taught him to set out calculations in a neat and orderly fashion, which was usually very difficult for young graduates.

When he left Dorman Long and went to Imperial College, almost the first subject in which he became interested was the use of models in structural analysis, in particular the Beggs apparatus. At that time, many practising structural engineers looked upon strain energy methods of analysis with considerable suspicion, and when simple model methods were available for checking calculations he and his colleagues thought it a good idea to write papers about this subject, so that engineers could really see that their strain energy calculations meant something. Indeed, one or two engineers whom he knew had installed model laboratories in their offices to save calculations.

We had now turned full circle. With computers we could calculate most highly indeterminate structures, and the greatest value of models lay in giving the solutions to problems which were either impossible or extremely difficult of solution by any other means.

Dr. Chapman showed slides which particularly illustrated the use of models in this way.

On the proposal of the President, a hearty vote of thanks was accorded the authors for their paper and for the very interesting slides they had shown in presenting it.

Mr. ROBERT FARNBOROUGH (Associate-Member), said he was impressed by the wide range of models presented. In all cases model analysis was applied to unconventional or at least unusual structures and economies in design were thereby often made possible. Was it not, however, possible that great benefit could be derived by subjecting more usual structures to the close scrutiny of model analyses. To illustrate with a simple example :—A slab behaves quite differently from the way which a conventional design normally assumes it to behave. And a beam and slab structure is still further removed from its ideal design equivalent. A recent test to destruction on such a building showed that there was a reserve of strength unaccountable for by usual design assumptions. Might not model analysis be the tool by which this margin between design strength and actual strength could be reduced and in this way benefit the unsophisticated structure to the same extent as the unusual ?

Mr. A. GOLDSTEIN (Member), said the authors had presented a paper which was not only very informative and interesting, but also one which, combined with the introduction of it, gave a great deal of scope for discussion.

He should explain that he was privileged as a young man to be taught by Professor Sparkes. It was very seldom that he had had occasion to disagree with his tutor, whose teaching he had assimilated in an earnest endeavour to do him credit.

Nevertheless, on this occasion he suspected that the authors of the paper had deliberately ' trailed their coats ' : and in view of that deliberate way of promoting discussion he felt sure they would not object if his remarks were somewhat critical.

In declaring his interest in this way he should say that, with the basic point that model analysis was advantageous, he found no reason to quarrel.

Having said that, he referred to Professor Sparkes' first illustration on the blackboard, and the suggestion made, quite rightly, that elastic analysis results might be ambiguous due to initial " lack of fit " of members. Professor Sparkes knew quite well that this difficulty could not be solved by models either.

Discussing the position of model work in this country and elsewhere, he agreed entirely that in places such as Lisbon and Italy there was very useful and exciting work being done.

We must remember, however, that commercial model testing had grown up in those countries for two reasons ; (1) that the testing laboratories could do the work quickly and to target dates, and (2) they could do it cheaply. The model maker in e.g., Lisbon was paid much less than what he would be paid here. If we were to go into the field of model testing for design purposes, as certainly the university laboratories and the Cement and Concrete Association were prepared to do, we should put our house in order and make it a thoroughly commercial proposition.

Mr. Goldstein then drew attention to two sentences at the beginning of the third paragraph of the author's Introduction which he particularly wanted to discuss ; he was sure that others would also have them in mind.

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1., *on the* 9*th March*, 1961, *the President, Lt.-Col. G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E., in the Chair. Published in " The Structural Engineer," Vol.* XXXIX, *No.* 3, *pp.* 85-99.

> " Model investigations normally result in structural savings, so that a further deterrent perhaps lies in the percentage system of consultants' remuneration, under which the achievement of an economical design, with its increased design costs and responsibility, is rewarded by reduced fees. This factor is, of course, partly offset in many cases where the client agrees to meet the cost of the model."

There were three things he wanted to say about this. First, if the statement were true—which he did not accept—then the fact that the client met the cost of manufacture of the model did not really alter the situation at all, because the cost of the pure manufacture of the model represented ' a drop in the bucket. ' The majority of the cost to any designer would be the time of the highly skilled engineers involved in operating on the model and studying results.

The second point was that model investigations normally resulted in structual savings. He did not see why it should be so necessarily. It was sometimes said that engineers' and designers' assessments always came out on the right side, but there was no guarantee that their results, with their ordinary methods of design or assessment, were necessarily on the right side. However, Mr. Goldstein agreed that in the majority of cases model investigations did result in savings.

The third and by far the most important matter in the two sentences he had quoted was the invidious and quite extraordinary suggestion which went to the nub of the remuneration of pretty well all the professions in this country—which he could not think was made with any other purpose than to trail the authors' coats and promote discussion—that a consulting engineer would be deterred from producing an economical design because if he did so his fees would be less. The remuneration of consultants was on the same basis as that of most other professional men. No consulting engineer worthy of the name would let that sort of consideration stand in his way. He hoped, therefore, that in replying to the discussion the authors would see fit either to substantiate their statement or withdraw it.

Mr. R. D. McMeekin (Member), congratulated the authors on their interesting and useful paper. All the problems except the last were problems of determining the behaviour under load of a structure of one material, i.e. steel, reinforced concrete, etc., but engineers often had to solve problems depending on the inter-action of two materials, and the most important of these was probably a raft foundation. There we had a case where the elastic properties of the raft affected the stress distribution on the ground beneath and, more important, where the properties of the ground affected the stresses and deformations in the raft.

If one had two identical rafts, identically loaded, but one was on clay and one on gravel, the stresses and deformations would not be the same. It was quite incorrect to take the load of the building, divide it by the raft area to find the uniform ground pressure, and then to turn the building upside down and design the raft as a floor. This seems obvious, but many raft designs ignore it.

The principal problem of raft design is that the load on the ground varies depending on the inter-relation of the elastic properties of the raft and the ground. All one knows for certain is that the sum of all ground pressures equals the weight of the building.

Was it possible to study that sort of problem by using models? Presumably the biggest difficulty would be to choose two model materials such that their ratio of elastic properties would be the same as in the building considered. It would seem that such a model could be made, and the stress distribution equated to the overall load of the building, so as to determine fairly accurately the stresses in the raft.

Mr. W. T. F. Austin (Associate-Member), asked if the authors would give their views on the application of the model technique to a type of bridge suitable for flyover junctions, particularly on urban motorways, where space was nearly always very restricted and the shape of the structure had to follow the lines of the road. This was a reinforced concrete slab supported directly on columns; it was particularly never square, very often tapered, and very unfavourable for analysis by mathematical means. It was not nearly so difficult to construct as it was to design; only the edge formwork was complicated.

He did not think the use of a computer was the right approach—it could require considerable time on a large one which might cost up to £2,000 a day to use. He thought the right approach was by model analysis, and he would be grateful if the authors would say whether or not it was so.

Carrying the matter a stage further, he did not think it was wise for the normal consulting engineer to conduct, himself in his own office, quantitive experiments in which constant temperature etc. were essential; the work should be put into the hands of an experienced laboratory. Professor Sparkes had mentioned laboratories in this country which were doing commercial work, such as the Cement and Concrete Association laboratory, for which Mr. Austin had a high regard. But Professor Sparkes, in omitting to mention the universities, had not done justice to himself or his own university. Commercial laboratories already existed in this country for hydraulic work, and Mr. Austin did not see any reason why similar laboratories for structural work should not appear.

Dr. Z. S. Makowski (Imperial College), referred to the statement made by the authors that the models should be as large as possible. He supported this point of view, because there were many factors, which sometimes could not be assessed accurately from experimental measurements made on small models.

For instance, it was impossible to produce a small scale model in which the influence of the semi-rigid connexions on the stress distribution could be truly represented.

It would also be found that it was virtually impossible to make a small scale model of a space structure with rigid nodes, in which the axial, flexural and torsional rigidities of all the members could be incorporated in the right proportions at the same time. Experiments on small scale models of such structures showed that torsion could exert an appreciable distorting influence on the stress distribution and that it was very difficult to assess it accurately. In such cases the size of the model had a great bearing on the accuracy of the predicted behaviour of the real structure. Unfortunately large models were rarely built because of high cost.

Recently Dr. Makowski had a rare opportunity to carry out on behalf of a commercial firm producing prefabricated steel structures a loading test on a model of a three-way double-layer steel grid. The span of the model was 56 ft.

There was a precise method of analysis available for this type of space grids, but certain coefficients, used in the numerical calculations, depended upon the flexibility of the connexions. A test on a small scale

model would not provide all the information required and a model of 56 ft. span had therefore been constructed.

The total load of 32 tons applied to the model consisted of steel scaffold tubes, spread uniformly over the whole area of the grid. The applied loading was twice the design load.

Strain measurements have been taken by means of Huggenberger extensometers. Cathetometers were used to determine the vertical deflexions of the nodal points of the grid.

The measured strains followed quite closely the stress pattern obtained by the analysis, though a much better correlation of results had been observed for the central part of the grid than at its boundaries, where some yielding of supports was detected under the load. The experimental deflexions were about 15 per cent smaller that those predicted by the analysis

Mr. B. BRAMALL (British Railways Laboratories) said he felt rather diffident, as a guest, about addressing the Institution because the laboratories had only just entered into the use of this method of testing models; but what they had done so far in that direction had opened up a field which it was believed was likely to become a considerable part of their structural research in the future. One model in particular which had given encouragement was designed and built to study a problem on the strengthening and stiffening of an existing bridge structure.

The bridge had steel girders and a very flexible deck. Tests on the actual bridge had proved that the stresses were not high in the girders, but the flexing of them was rather considerable and the rails themselves were subjected to intense flexing. It was therefore essential to get to grips with the problem to improve conditions; some longitudinal stiffening was desirable.

Basically they could deal mathematically with a series of rails as beams. The problem was simple enough when dealing with one beam on a foundation, not affected by any other beam. In this case, of course, there were four rails, each having an influence on the others. There again the mathematical solution was not too difficult. But there was the element of the stiffness of the deck longitudinally, and even the contribution of the ballast, which made it very nearly impossible to arrive at a purely mathematical solution.

Mr. Bramall then illustrated his remarks with a lantern slide showing the section of the main span. In each dip of the floor there was ballast.

Another slide showed that the stiffening member could be a girder within the space between the two tracks, which would have been the simplest course from the practical point of view. It was also thought there might be a hope of stiffening by sandwiching the deck between upper and lower flat strips. The imperfectly triangulated construction thereby produced was thought to be worth while trying with models, but the results were disappointing.

It was suggested also that the effect of girders between the tracks might be improved if the web were serrated and connected to the top and bottom of the transverse troughs. That was tried; but any gain was so small as not to justify the elaboration.

The next slide showed the members used in the model to represent the plain girder, the castellated girder and the flat strip.

A problem with the miniature girders was that if they had the joist section milled out or fabricated, the stringer would have a very very thin web and would be liable to buckling in the model. The web was made out of perspex, the ratio of the perspex to the aluminium being about 10 : 1.

Mr. Bramall then illustrated the results of comparisons with each arrangement, and a cross-section of the bridge deck. Gauges were placed on the deck to measure the curvature at the rail positions. The graphs showed the comparison between the various stiffening arrangements, of which 14 were tried. Obviously it would be very expensive to try full scale alternatives.

In the next illustration the model was mounted on a grinding machine base with gauges located underneath.

The application of load to the rails was then illustrated and the next slide showed two girders mounted side by side on the model.

A point of detail was that we did not want interaction between the rails and the deck, we did not want them to act together as a beam transmitting shear. So that the connexion was made by small pivoted links at the position of the sleepers.

Curvature diagrams relating to a beam with continuous elastic foundations were then shown. The top diagram related to the normal conditions; in the second one the support stiffness was doubled; and in the third one the stiffness of the longitudinal itself was doubled, but the supports maintained their original stiffness. This showed simply the benefit of longitudinal stiffening rather than stiffening the support. Support stiffening would have meant stiffening the deck laterally.

To the layman, concluded Mr. Bramall, the first approach to tackling a problem by the use of models might well be that it was a substitute for mathematical treatment. He stressed the point that the mathematical approach assisted in indicating how we should tackle a given problem. In dealing with the problem he had been concerned with, and had illustrated, the limit of the mathematical treatment had been reached, and the model analysis went on from there.

Mr. S. B. ZUKAS (Associate Member) commented that on this occasion the authors had referred to models for specific problems and he wondered to what extent they had found it possible to generalize. He also wondered to what extent it had been proved that a specific model represented the real structure.

He was surprised to see in the moiré photographs such detailed representation of the curvature around openings in flat plates. He had recently tested a full-size model of a perforated concrete flat plate panel and found that the curvature of the perforated panel was little different from that of a similar but unperforated panel which had been tested earlier.

Mr. F. N. SPARKES said that he thought that the authors had unwittingly conveyed the impression that the use of models to study structural problems was a fairly straightforward matter. Mr. Sparkes had had a good deal of experience in the use of models during the war and fully supported the authors' contentions (a) that they enabled some problems to be solved which were otherwise insoluble, and (b) enabled the effects of altering variables to be studied fairly simply.

Nevertheless, there were certain snags in the use of models, and he thought it was important that engineers should not regard this technique as one which could be applied by any engineer without special knowledge and experience. The authors were clearly aware of this and it had also been brought out in the discussion by reference to the need for specialist consultants to handle this type of work.

One speaker had already referred to the problems of using models to represent materials having different elastic properties. Another difficulty was concerned with the gravitational effects of dead-loads which could not themselves be scaled down, as could the dimensions of the structure. This gravitational effect was probably present when the dam referred to in the paper was tested to failure, as shown in Fig. 18. The cracks were formed by displacement of part of the dam and the amount of displacement would be conditioned by the dead weight. Experience during the war had shown that such displacements were the same on the model as on the full scale. Thus, to get a true model representation of the full-scale damage would seem to require that the displaced part of the dam should be restored to such a position as was appropriate to the scale of the model—in other words the cracks shown in Fig. 18 should in fact be only 1/50th of the size shown to get a true representation.

These remarks did not in any way invalidate the value of the work done on the dam by the authors, since the question of actual failure was rather a hypothetical one and in any case, presumably only of secondary interest. Nevertheless, they do point to one of the snags which an inexperienced user of models might overlook.

Another aspect of the use of models concerned the importance of study of dimensional analysis. This is a subject which does not normally concern engineers in their day-to-day work, but it was most important that the fundamental factors affecting the use of models should be understood by those who used them. It is simple to scale down dimensions but it is not always so easy to decide whether to scale down other physical or mechanical properties such as strength, elasticity, frequency and amplitude of applied vibrations, temperatures and so on.

One speaker had raised the question as to whether structures made of a composite material such as reinforced concrete could be studied by using models. Mr. Sparkes had tested many bridges of 1/10th and 1/4th scale and of the girder, arch and bow-string types in reinforced concrete and the tests had been quite satisfactory.

While fully agreeing with the views of the authors as to the great value to be derived from model tests, Mr. Sparkes thought it would also be of great help to engineers if the authors could indicate the snags and difficulties in their use, some of which had been referred to above and by other speakers.

THE PRESIDENT said he was rather concerned by the warning which Mr. Austin had given against the use of models by the staffs of ordinary engineering organisations, and felt that it should not be left without further comment. He asked if the authors would support the view that the model could be of profound value in demonstrating points of behaviour, such as the type of deformation, provided one knew always at the outset the solution that one was looking for. The model was not used to give the solution, but to indicate the direction in which the solution would be found, to indicate to the engineer the tests he was required to make.

Written Discussion

Mr. D. ALLEN (Associate-Member), commented that as an engineer with the designers of Ganga Bridge he had made estimates of the closure forces, and later as an engineer with the fabricators and erectors of the bridge he had had the opportunity to use the model of Ganga Bridge and to compare the closure forces with those actually measured.

He had two main criticisms of the model itself:

(a) The hangers which support the railway deck from the central gusset were not incorporated in the model. The presence of these hangers affects the closure forces of the second lower diagonal. As the hangers were also 'cambered,' (i.e. shortened to account for dead load and half live load extensions) this had the effect of cocking-up the bottom cord.

(b) No account can be taken of the self weight of the members which can produce deflexions and end rotations of significant magnitudes. The model would give an accurate measure of the closure forces of a truss erected on its side, as for example in a trial assembly.

It would be of interest to give the erection sequence in a typical mid-span panel which would perhaps illustrate the problems of constructing a model that was to conform to the actual bridge. In general, once a member had been pinned at either end there was no great trouble in fairing all the holes using the split wedge drift pins designed by R. J. C. Tweed (see Fig. 13 of Reference 29) and thus imparting initial bending into the members (prestressing).

Sequence

(1) Bottom chord with gussets, erected and supported by adjustable wire ropes from the top chord gusset.

(2) Hanger erected.

(3) First lower diagonal erected with the central gusset preriveted to it. There was no trouble in pinning both ends because of the great flexibility of the hangers.

(4) First upper diagonal erected. The wire rope supports which had a jacking and screw device were adjusted to allow both ends to be pinned.

(5) Second lower diagonal erected. Generally the lower chord gusset had to be forced down, but the holes were initially open enough to allow the insertion of a split-wedge drift.

(6) Second upper diagonal was cantilevered from the central gusset.

(7) Top chord with gussets erected. The chord joint was made first with the chord supported by the crane. Then the end of the cantilevered diagonal was pulled up towards the chord with a 'pull-lift,' at the same time lowering the chord till the diagonal connexion could be made.

The lower end panels gave the most trouble to erect because here the secondary movements were highest. It was necessary to bend the end post in towards the span and this was done most effectively by first completing the post-lower diagonal joint and then letting the cantilever deflexions of the diagonal and post bring the lower joint together. Even after this 'dodge' it was still necessary to use a pull-lift to fair the holes sufficiently to insert a split-wedge drift.

Although the model had limitations it did give the erectors a fair indication of the order of the closure forces, as well as giving a guide to the effect of varying erection sequences.

Reply to the Discussion

The authors agreed with Mr. ROBERT FARNBOROUGH that slab problems were suitable for model analysis. Models of perspex or resins (Reference 33 in the paper) could be used to obtain the elastic stress distribution under different load systems. The model could be machined or moulded and strains measured with electrical resistance strain gauges. A second method would be to make scale models in cement mortar, and information could be obtained on the behaviour of the prototype up to failing load (References 19 and 26). Such a model could certainly exhibit the actual mode of failure.

The authors were most stimulated by Mr. A. GOLDSTEIN'S spirited contribution. The simple illustration given on the blackboard to show the importance of 'lack of fit' in elastic analysis was not intended to indicate that models overcame such difficulties—rather, it was emphasised, and it is very important to appreciate, that it is extremely difficult to obtain agreement between elastic analysis and actual behaviour of redundant structures. It is a salutary lesson to see how it is almost impossible to get answers that agree from several models of a particular redundant structure which are intended to be identical. In this case, the purpose of the model is to give a better understanding of the significance of some of the assumptions underlying the analysis.

The authors agreed that probably the main reason why commercial model testing had developed in other countries was that the work could be done quickly and to target dates. It must be admitted that normally universities are asked to do problems of which the solution and probably model technique has not been known, and in these circumstances it is extremely difficult to forecast how long the work will take and how much it will cost. The examples given in the paper, however, are all of work which was completed within the target dates and probably at much less cost than if the work had been done in commercial laboratories abroad. In referring to the cost of the work, the authors included the cost of materials, the making of the model by skilled technicians and the fees for designing the tests, carrying out the work and preparation of the report.

It was never the authors' intention to give the impression that a consulting engineer would be deterred from producing an economical design 'because if he did so his fees would be less.' In the paper the authors were pointing out that there was an anomaly in the fact that, since most model investigations result in saving in structure costs, the engineer receives less fees for the greater trouble he has taken. In saying that something 'was a deterrent,' the authors were not saying that it 'deterred.'; All the models described in the paper provided clear evidence that this 'deterrent' did not, in fact, deter British engineers. If British engineers did not have model work done it was for the reason that they had not known of the facilities available and of the great value of model results. The authors agreed that a consulting engineer would not be deterred from producing an economical design, because if he did so his fees would be less, and the examples in the paper confirm this view.

In reply to Mr. R. D. MCMEEKIN, a lot of work had been done by the Italians in simulating elastic properties using special aggregates—e.g. pumice aggregate—and they could produce materials with a predicted modulus with facility over a wide range. Laboratory skill in this work could certainly be built up in this country to deal with the type of problem which Mr. McMeekin described.

The authors agreed with Mr. W. T. F. AUSTIN that a model approach to problems of bridges which were really skew reinforced concrete slabs supported on columns was particularly suitable. Indeed, the paper by Finch and Goldstein (Reference 27) deals with such a problem. A recent investigation was carried out at Imperial College using a cement mortar model of such a structure at the London Road Station, Manchester.

Mr. Austin was certainly wise to appreciate that the design, making and testing of models is a very specialist business, and although many engineers are used to making small qualitative models considerable skill was required before reliable quantitative results could be obtained. It is true that facilities and experience do exist in some universities for structural model work, and the authors hoped that consulting engineers would use these facilities in the universities, which are very conveniently available in many different parts of the country.

The authors were interested in the example of model testing given by their colleague, Dr. Z. S. MAKOWSKI. Information from tests such as he described, besides being vital to the engineer on the particular problem, also gives experience on the behaviour of structures under practical conditions.

Mr. B. BRAMALL'S contribution laid stress on the value of models in giving information which could not be obtained in any other way. This, indeed, is largely the theme the authors wished to emphasise in the paper. They agreed that a basic understanding of the problem was necessary to plan and interpret model tests. They would also add that the experience gained in performing a model test added greatly to this understanding.

In reply to Mr. S. B. ZUKAS, although examples had been given of the solution of particular problems by the use of models, the method could also be used for general investigations. Models were currently being used, for example, at Imperial College for a general investigation on composite beams.

The extent to which a particular model represented a real structure depended on so many factors that it was not possible to give a definite answer. If, however, a model analysis and a theoretical analysis are based on the same assumptions, and if the actual structure is also based on these assumptions, then the model will represent the real structure. Mr. Zukas' reference to the full-size model of a perforated concrete flat plate panel is very interesting, but it was not possible to make any comments on his observations, as no details of the work have been given.

The authors were very pleased Mr. F. N. SPARKES had emphasised the need for specialist consultants to handle model analysis. In illustrating the use of models in problems of arch dam design, the authors had been concerned only with the stresses in a dam due to water load. Certainly the cracks and stresses in the dam would be very much influenced by the dead weight, and a true representation of the behaviour of the dam would have to take account both of water load and dead load and indeed other factors such as temperature effects and creep. The ability of models to portray the behaviour of full-scale structures needs careful study, and many publications dealing with this problem are referred to in the bibliography at the end of the paper.

The authors were interested to hear that the tests on the 1/10th and 1/4th scale bridges in reinforced concrete had been quite satisfactory. At Imperial College, too, they had made a number of tests on composite beams and had systematically used models of different sizes in order to see what correlation they could achieve between the model and full scale results. In general, it could be said that better correlation was obtained than some may have feared, having regard to the difficulty of scaling down concrete.

Mr. Sparkes' suggestion that notes on the snags and difficulties in the use of models would be of great help to engineers is certainly true, but these problems would really be the subject of a separate paper. It would not, however, be a replacement for the skill and experience of the specialist consultants, to whom Mr. Sparkes had referred.

The authors are very grateful for the explanations given by Mr. D. ALLEN on the sequence of operations on the Ganga Bridge.

It is very encouraging that Mr. Allen, who has been at the receiving end of a model investigation, has written to give his experience. It might almost be considered in the nature of an unsolicited testimonial!

Book Reviews

Reinforced Concrete Chimneys, by C. Percy Taylor and Leslie Turner. Second Edition revised by Leslie Turner. (London : Concrete Publications, 1960). 9 in. × 6 in., 81 plus vi pp., 12s.

This book, which deals in a clear manner with the theoretical and practical questions relating to the design and erection of reinforced concrete chimneys, has been revised by Mr. Leslie Turner to include up-to-date methods and examples of more recent chimneys. Reference is also made to the U.S.A. standard specification for reinforced concrete chimneys.

In the new edition an improved method of analysing annular sections subjected to bending and direct thrust replaces the more complex method given previously, and a new chapter has been added dealing with the deflexion and sway of tall chimneys due to the effects of wind. Some of the design data, such as the combination of thermal and structural stresses, are also applicable to such structures as concrete tanks and biological shields. As in the first edition, an example of the complete design of a chimney is given.

Structural Concrete, by Kurt Billig. (London : Macmillan, 1960). 8¾ in. × 5¾ in. 1,018 plus xx pp., 84s.

This new work by Professor Billig is a continuation of his two previous books " Prestressed Concrete " and " Precast Concrete " published a few years ago. These previous volumes dealt with specialist branches of structural concrete but the aim of the present work, we are told, is to provide a minor encyclopaedia on " Structural Concrete " within a single volume.

Like most text books of similar nature which try to be comprehesive and all-embracing, it is impossible to deal adequately with any particular subject but it does give a very good outline of the subjects involved and a comprehensive list of references. In this connexion it might well be used as a sort of general text book for 1st and 2nd year students at Universities and Technological Institutes.

There are several errors which will no doubt be rectified in a second edition—for example, Table 1 appears to be based on B.S.12 : 1947, although B.S.12 : 1958 is quoted in reference. In the latter there is no test for residue, the fineness being dealt with by specific surface. The initial setting time is misquoted and the strengths shown for mortar compression tests are inaccurate, but perhaps what is more important, no reference at all is made to the new 4 in. concrete cube tests.

Again, on page 16 there appears to be some confusion in that the compounds C_3S, C_2S, C_3A, and C_4AF are referred to as formed during the setting of the cement. These brief references are sufficient to confirm the point made above that this work requires revision and bringing up-to-date in some respects, but when we have said that it must be emphasised that this book is much better arranged than his two previous works, and, in fact, it is Professor Billig's best book so far.

Engineering Design with Rubber, by A. R. Payne and J. R. Scott. (London : Maclaren, 1960). 8½ in. × 5½ in., 256 plus ix pp., Illustrated. 50s.

Realising the increased use and the rapidly developing interest in the use of rubber in engineering, the Research Association of the British Rubber Manufacturers organised a series of technical symposia upon which " Engineering Design with Rubber " is based.

The first half of the book deals with the fundamental engineering properties of rubber and the remaining chapters deal adequately with the theoretical aspects of these properties in relation to engineering usage. Contributions from leading experts cover the usage of rubber in the civil engineering field ; in the design and development of the material as anti-vibration mountings ; resonance and transmissibility ; practical design details and a useful chapter on the collaboration necessary between engineers and rubber technologists.

Although there is no great emphasis made on its application to structural engineering, the background material will be useful to structural engineers, and should give rise to an increased awareness of the possibilities of the material.

Each chapter has an ample bibliography and the appendices cover British and American standard tests for both natural and synthetic rubber and useful tables on the general properties and character of the various rubber types.

It is regretted that there is little guidance on the abrasion-resistant qualities of the material and neither is mention made of possible future applications, however, it is felt that although this book is not primarily intended for the structural engineer, it should prove useful and stimulating to many in the profession.

D.R.R.D

Aluminium Strut Design*

Discussion on the Paper by J. B. Dwight, M.A., M.Sc., A.M.I.Mech.E.

Mr. DWIGHT said that in the paper he had attempted to reduce the complication and to make life easier for the designer. Aluminium had two serious disadvantages which were rather like two millstones around the neck of the designer in aluminium—first the high cost of the material and secondly the low E. Aluminium cost about two and a half times as much as steel and it was three times as flexible. But there were two important advantages — first that it did not rust, and secondly we had the marvellously versatile extrusion process at our disposal. Because it did not rust, we had not to worry about using thin section; we might use $\frac{1}{8}$ in. section, whereas in the case of steel the $\frac{1}{8}$ in. might represent the rust allowance.

THE PRESIDENT conveyed to Mr. Dwight the very sincere thanks of the meeting for his excellent presentation of the paper.

Mr. MORRIS T. SHAW, M.I.C.E., (Member), congratulated the Author on an excellent paper and referred to a statement he had made several times in the lecture, "that aluminium alloys did not rust."

He recalled that about five years ago when visiting Fort Halstead he had seen evidence of "layer corrosion" in certain aluminium alloy bridge panels.

The nature of the attack was very much akin to that in ferrous corrosion and Mr. Shaw wondered if Mr. Dwight had met this problem and if he knew what answer industry had supplied.

Mr. DWIGHT said the paper was concerned with aluminium strut design and he felt that corrosion was a little outside his orbit on this occasion. Nevertheless, he would like to say what little he knew about it.

When speaking of structural aluminium design he was thinking of what was virtually the standard type of aluminium used for fabricated structures, the type he had illustrated. It had never been given a name in this country, and was referred to by its specification number H 30-WP. It was a very different type of aluminium from that with which aeroplanes were made. The bridge panels referred to by Mr. Shaw may have been in high tensile "duralumin" type of material, in order to achieve the ultimate in weight saving. If this were the case there might well be a risk of corrosion of a kind not met with in the H 30 alloy.

Mr. S. R. BANKS (Member) said that as Mr. Dwight had indicated, this paper gave us a foretaste of the revised Report on the Structural Use of Aluminium, which the Institution had been preparing for four or five years. He hoped the paper did anticipate an early presentation of that Report—in other words, that the main dish would follow soon.

Complimenting Mr. Dwight on an admirable brief text on aluminium strut design, he felt sure that the Institution's Report, when it came out, would be the better for having this background document. He hoped the other parts of the Report would also be backed up by similar papers presented to the Institution, particularly in regard to eccentrically loaded columns as distinct from the axially loaded ones, and also the more complex subject of combined bending and compression.

Again, the paper contained some very clear statements about factors of safety and showed us the difference in approach for tension and compression members. Whilst he was not in any way criticising Mr. Dwight's careful examination of the several strut formulae—it led to better understanding of column-action and enabled the engineer to use convenient concepts such as that of the equivalent slenderness ratio—he pointed out that strut formulae in the end all seemed to depend on actual tests of struts, and that if a formula did not fit, it had to be doctored until it did fit. In fact, the designer, although he might appear to be using a permissible stress based on a proof stress to design his columns, was actually applying a safety factor of two against the failure load.

To the engineer, moreover, it did not matter that proof stress in tension might not always be the same as the proof stress in compression.

Regarding tension, the author had brought out the point that permissible stresses were based not only on the proof stress, but that an apprehensive eye must also be kept on the ultimate stress. Thus, for H 30-WP there was rightly a comparatively high safety factor of 2·3, whereas for mild steel or an aluminium alloy such as N6 the safety factor could be reduced to 1·6.

The speaker expressed indebtedness to Mr. Dwight for having pin-pointed (on p. 59) the several lines on which further research would be appropriate. He reassured Mr. Dwight on this by pointing out that the aluminium industry, through the Aluminium Development Association, was aware of the need for more research and had in fact for some years sponsored it at the universities. The bulb angle sections which had been standardised or partly standardised by the A.D.A. were of two shapes, one being applicable to all equal angles and one to all unequal ones. In fact only four constants for angles were needed. One, the shear-centre location of the equal bulb angle, had been obtained at Birmingham University with an electric resistance analogue, and it was expected that the other three would follow soon.

There was one other matter in connexion with research. It was the fact that, however good our strut formula, and however precise the permissible stresses, the designer had still to make his own guess at end-conditions in order to design a strut in a framework. The difficulty applied to steel design as well as to aluminium. Several investigations were being made at several universities on the interactions of members in frameworks, and it was hoped that this research would go at any rate part of the way to removing some of the guesswork.

Mr. DWIGHT thanked Mr. Banks for his helpful remarks and was glad he had mentioned the A.D.A. sections (Fig. 18). These formed three ranges of section, each of a standardized geometrical shape, so that when the constants for each range had been determined we

**Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on the 9th February, 1961. Lt.-Col. G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer" Vol. XXXIX, No. 2, pp.47-61.*

could use any of the sections with confidence. It was interesting to hear that the shear centre location for the equal bulb angle had been determined with the aid of an analogue at Birmingham, as the calculation of this by ordinary means was very difficult.

Concerning the effective length, he said we had been debating whether to use the tangent-modulus or Perry-Robertson formula, but that really the difference between them became unimportant when one considered the far greater errors inherent in the guessing procedure for the effective length. Methods of accurately calculating the elastic behaviour of a strut in a framework were available allowing for inter-action with the other members and even taking terminal buckling into account, but they were very laborious to apply. Moreover they did not look after inelastic effects. Recently he had come to feel that we were soon going to enter an era in which the idea of slenderness ratio would to some extent give way to that of the " squash load " at any rate in steel. It was difficult to see at present how the new ideas would be applied to aluminium structures.

Mr. H. TOTTENHAM asked for further information on two points. Firstly—torsional bending could occur with tension in a member, as can be shown both theoretically and in the laboratory; had this effect been noticed in aluminium structures and this factor been considered in the revised Report on the Structural Use of Aluminium.

Secondly—eccentric compression in struts; the paper dealt only with central compression, the problem of eccentric compression was more complex and Mr. Tottenham wondered whether it would be possible to treat this in the same sort of way in the design recommendations.

Mr. DWIGHT confessed that the possibility of torsional buckling occurring in a tension member had not occurred to him; indeed, he could envisage it happening only if it were eccentrically loaded.

He had never come across it in an aluminium structure, but it was conceivable that it could occur in a thin-walled member if the load were sufficiently offset. It was really a case of combined bending and axial load, and was related to Mr. Tottenham's second question.

The treatment of combined bending and axial load, or eccentric axial loading which was much the same thing, was a wide subject on its own. He had not included it in the paper because he had felt that he had had enough material to tell one story. It had been covered in the revised Report on the Structural Use of Aluminium.

Dr. M. S. G. CULLIMORE (Associate-Member) reinforced Mr. Dwight's remarks in the paper about the use of the equivalent slenderness ratio approach for obtaining the critical stress by plastic buckling. It was an engineering method, it was delightfully simple, and in recent tests on channel sections he had found that it did in fact give very good results.

Equation (5) on page 51, for a section having one axis of symmetry :

$$(p - p_v)\left\{\propto p^2 - p\,(p_u + p_t) + p_u p_t\right\} = 0$$

was a quadratic equation for p, and what we were in fact doing in deriving the plastic torsion flexure failure stress by using the device of equation (10) on page 56 :

$$f = \frac{E_t p}{E}$$

was to reduce the roots of equation (5) in the ratio E_t/E.

This implied that the critical stress for the basic modes, flexure p_u and torsion p_t, were both reduced in the ratio E_t/E.

Obviously this gave the correct answer for the flexural mode p_u, but less so for the torsional mode :

$$p_t = \frac{GJ}{Ip} + \frac{\pi^2 EH}{IpL^2}$$

Where the first term was predominant in sections with low warping stiffness, the reduction in p_t due to E being reduced to E_t would be small, as G diminished less rapidly than E for moderate plastic strains. Thus the true value of p_t would probably be greater than $(E_t/E)p_t$, and hence the use of equation (10) would give a value somewhat less than the true value.

Where the warping term was high compared with $\frac{GJ}{Ip}$, in cases such as the channel, particularly at the shorter lengths, the reduction of p_t in the ratio E_t/E should give a value much nearer the true value; and the calculated value of p, although still below the true value, should give a good answer.

This was borne out by R. E. Smith's test results (Ref. 18).

Mr. DWIGHT said it was an interesting point that the e.s.r. approach when applied to terminal buckling would tend to be more conservative for an angle than for a channel. It would be interesting to know, however, just what was the reduction below the elastic value for the extreme case, that is for a section such as an angle in which there was very little warping. A lot of work had been done on inelastic local buckling, which was a very tough problem academically, and some progress had been made, although he did not think it made much difference to the designer. So far as he was aware, there had been very little work on inelastic torsional buckling.

Dr. P. S. BULSON (Associate-Member) spoke about two points Mr. Dwight had mentioned in his paper.

The first was local buckling of a thin walled square tube. The slight inward movement of the plate junctions in the buckled mode, which had been observed in tests, could be taken into account in the analysis.

In preparing equations representing boundary conditions at the junction, it is usual to assume no outward or inward bending of the edges of the component plates.

If, however, an inward deflexion is allowed and taken equal to the lateral shortening of the adjacent plate, a more correct analysis is possible. The difficulty here is that, because lateral shortening is a second order effect, the set of simultaneous equations cannot be solved in general form, so that the method is not suitable for rapid design calculations.

Secondly, Mr. Dwight drew attention to the need for further research on the local buckling of lipped sections, with special reference to the ratio of lip to flange width at which the mode of buckling changes. There is, of course, a transition from one mode to the other, and it is possible to obtain a theoretical expression for the transition curve for a particular case. However, an elaborate computation is involved and the theory is difficult to apply when the plate buckles into more than one half wave. A simplified or approximate approach is needed, and might be found by concentrating on compatibility of wavelength between the lip and the plate.

He agreed with Mr. Dwight's suggestion that further research be carried out.

Mr. DWIGHT thanked Dr. Bulson for the information he had given. He thought the first point was interesting academically, but he would like to know what was the effect on the buckling load.

Dr. BULSON said the effect of the inward movement was small.

PROFESSOR SIR ALFRED PUGSLEY, O.B.E., (Past-President) said that in his view Mr. Dwight seemed to have attributed a little too much to America. Our knowledge of torsional buckling derived first from Germany. Such buckling had been experienced in the airship world, where preliminary theories were developed. There was then plenty of experimental evidence on the practical side, but the first experiments on pure torsional instability in this country were made by Thomas, of Battersea Polytechnic, around 1939. He had endeavoured to avoid any warping restraint.

The importance of different types of instability that can arise in a strut had been emphasised by Mr. Dwight; one would like to know his views on the degree of catastrophe involved in failure due to the various types of buckling. A thin rectangular section tube need not collapse at the onset of local buckling; one could go on imposing more load.

He did not know whether it was fair to say so, but he was rather shocked that the younger generation of engineers interested in aluminium alloys had not shown their independence of steel strut history in regard to the use of the Perry-Robertson formula; they so obviously really preferred the tangential E approach.

Finally, he felt that anyone not accustomed to very thin sections might be a little afraid of their behaviour under more or less dynamic conditions; he could assure any such that experimental work under impact conditions at vehicle speeds, in hand at Bristol, had so far given confidence in the adequate compression behaviour of short tubular struts under sudden loading.

Mr. DWIGHT thanked Sir Alfred for having raised some interesting points and said he felt some diffidence in dealing with them because Sir Alfred's experience in this field was far greater than his own.

He was not aware of the work done by Thomas at Battersea just before the war. But he could recollect a paper by Sir Alfred on the torsional buckling of an angle; perhaps his experiments were made previous to those of Thomas in 1939.

SIR ALFRED PUGSLEY said his very elementary theory was published in 1932.

Mr. DWIGHT, turning to Sir Alfred's second point said that the degree of catastrophe involved with the different types of buckling was a major issue. There were two ways of looking at it. One was to say that any degree of buckling was a bad thing and adopt a definite load factor based on initial instability. The other was to think entirely in terms of ultimate collapse. For column and torsional buckling the two approaches amounted to much the same thing, but with local instability the onset of buckling did not necessarily indicate catastrophic failure of the strut, especially with very thin sections. Thus with local buckling it was logical to base design on maximum load, rather than the initial buckling, as was now the practice in light gauge steel.

With aluminium structures built of extrusions, however, the sections used were just that bit thicker, and the reserve of strength after buckling was correspondingly less. For this reason it had been decided, in drafting the Institution's Report on Structural Aluminium, that the extra complication of taking part-buckled behaviour into account was not warranted and that calculations should be based on the theoretical initial buckling load.

It was interesting to hear that resistance to local buckling was improved under dynamic conditions.

Referring to the continued survival of the Perry-Robertson formula, Mr. Dwight agreed that, knowing the theoretical justification for the tangent-modulus formula, there was a strong argument in favour of introducing the latter. At the same time it did not make much difference which formula one adopted when one took into account the unsatisfactory effective length position, and the Perry-Robertson formula had the merit of being more flexible to use. If a new version of an alloy were introduced, it was a simple matter to construct an appropriate Perry-Robertson curve. He admitted that a similar argument could be applied to the Alcan straight-line construction which appeared to give remarkably close agreement with the theoretical correct tangent-modulus curve.

The PRESIDENT remarked that he was one of those who had suffered originally under Euler, then Robertson and, later, Perry-Robertson, and being employed by steelwork people it had meant varying tables from time to time as the formula was modified.

He felt that the meeting should take Sir Alfred's very wise advice and leave it, at this late stage of the evening.

He thanked Mr. Dwight, on behalf of the meeting, for the very excellent way in which he had dealt with a variety of questions. The President added that he was delighted in a number of respects. During recent months we had heard a great deal of reference to "the wind of change." He felt that the wind of change had entered the lecture room that evening particularly when Mr. Dwight had spoken of the "squash load," for he could not remember having heard a phrase more delightfully apt; he would use it when talking to his own colleagues in the office in terms of plastic loading. The phrase indicated exactly what we wanted to convey.

Corrigenda

Page 48, *Col.* 1. Last paragraph but one above the heading "Load Factor"; "... while column buckling, if present, will appear for struts of intermediate length." This should be 'torsional' buckling.

Page 49, *Col.* 2. (*top*). Empirical stress-strain formula should have another sigma at the end.:

$$\varepsilon = \frac{\sigma}{E} + A \sinh B\sigma$$

Page 50. *Fig.* 4. The curves refer to *H*20-*WP* (not *H*30-*WP*). The author explained that the two materials are very similar.

Page 59, *Fig* 21. The dimension

$$\left\{\frac{N}{2} + 1 + \sqrt{3(N+1)}\right\} t$$

should read

$$\left\{\frac{N}{2} + 1 + \sqrt{3(N-1)}\right\} t$$

Institution Notices and Proceedings

THE PRESIDENT'S SPEECH AT THE ANNUAL DINNER, 1961

The Annual Dinner of the Institution of Structural Engineers was held at the Dorchester Hotel, London, W.1., on the 5th May, 1961.

After thanks for the Toast "The Institution" proposed by the Rt. Hon. Lord John Hope, the Minister of Works, the President, Lieut.-Colonel G. W. Kirkland, continued :—

"Our Institution is unique in that we are the sole professional body in the world representing engineers in the specialised field of structures. Our 9,000 members may not represent a high proportion in the whole field of professional engineers but our importance as specialists is self-evident. We are naturally jealous of our position and would resist any association or amalgamation which might lead to the loss of our identity.

I undertook a little research into the numerical strength of the engineering profession and I find that there are about 150,000 engineers, members of Chartered Engineering bodies. Allowing for plural memberships this figure may perhaps be reduced to about 100,000. There are many other engineers, qualified by examination and experience, and members of bodies other than Chartered Institutions. Yet another great body of men exists qualified by University degrees, and experience in the field of engineering, who are not members of any recognised engineering society. The overall total may well approach a quarter of a million.

Can we not claim then that the profession of the Engineer is the largest in the country and take some steps to ensure that in this age of the Scientist and Technologist, the professional engineer is accorded his rightful place?

Our education and training is as intensive and as rigorous as many of the other professions, and yet our status amongst the population, in all fields of public life and even amongst ourselves, is nothing like as high as would appear to be warranted by our contribution to Society and our numerical strength.

The medical profession through the B.M.A. represents almost 80,000 doctors practising in branches as varying as those in the field of engineering, covering gynaecology, pathology, psychiatry, neurology, endocrinology, and many others, as well as general practice —and no profession has a higher status.

Chartered Accountants number almost 43,000 including their Irish and Scottish colleagues and everyone knows of them and accords them their rightful place.

The Architects now number 25,000 members of the Royal Institute and Mr. Everyman and his wife certainly know of the Architect's role and acknowledge his work and contribution to Society.

Why then do we Engineers fail? And what can be done in an endeavour to rectify the situation?

I believe the failure to be our own fault, partly through absorption in our own subjects and through the scatter effect of the large numbers of bodies which represent us.

And to rectify the position? I suggest those of us—so entitled—might start at once by describing ourselves simply as Chartered Engineers; after all, our field of interests can readily be ascertained by reference to our professional qualifications. We would number 100,000 then, and the Chartered Engineer might well become of some account.

An additional way would be by having an organisation to represent us, not in our specialised fields, but a body to secure proper recruitment to our profession, and to ensure our proper representation in the councils of the world.

I do not advocate the formation of yet another engineering institution, since there exists a body whose role this should properly be. It will, however, require much patience and persuasion to obtain within the administration of our "Mother" Institution that change of heart necessary to divert her effort from "take-over bids," and examination of candidates at Finals level in the specialised subjects more than adequately covered at an even higher level by other Chartered Institutions.

Let the "Mother"—the Institution of Civil Engineers—give encouragement to her various offspring, let her urge and help those bodies of non-chartered engineers whose standard of education, training and professional status does not yet reach the required level, to attain the requisite standard, and then admit the body in its own right on the basis of proportional representation to her Council and ultimately grant her corporate members the individual honour of the title of Chartered Engineer.

Persuade her to use her prestige, influence and strength to the direction of the great forces of nature *in man* for the advancement of the whole science and art of all branches of Engineering, and for the betterment of the knowledge and status of the professional engineer.

I feel certain that her larger children, the Institutions of Mechanical and Electrical Engineers, would give their support to so worthy an end.

For ourselves, we *are* Chartered Engineers and are proud of our Institution and our profession. On behalf of all our members, I thank you again, My Lord, for the Toast, and you, our friends and guests, for your kindly reception of it.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1. on Thursday, 22nd June, 1961 at 5 p.m. Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E., in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

AL-JANIBI, Ziad-Nasir, of London.
ARNOTT, Peter Bruce, of Salisbury, Southern Rhodesia.
BROOKE, Richard Gladstone, of Huddersfield.
CULL, Denis Augustine John, of London.
DAVIS, John Michael, of Nairobi, Kenya.
HANNA, Paul Eric, of Bristol.
HEARSEY, George Gerald, of London.
HOLLAND, John Kenneth, of Belfast, Northern Ireland.
KNOWLES, Antony Paul, of Oldham, Lancs.
LIM CHENG KIAT, Paul, of Singapore.
MARCUSSEN, Einar Wilhelm, of Salisbury, Southern Rhodesia.

MOHEBBI, Hassan, of London.
ROTHWELL, David, of Bolton, Lancs.
SIMPSON, Ralph James, of London.
TAYLOR, David, of Bolton, Lancs.

GRADUATES

AL-BAYAH, Ismail Ibrahim, of Rheinhausen, Germany.
AMIN, Hariprasad M, of Beeston, Leeds.
BARNARD, David Philip, B.Sc., of South Shields, Co. Durham.
BASU, Kalyan Kumar, of Glasgow.
BRADBURY, Arthur Basil, M.Sc., of Radcliffe on Trent, Nottingham.
BROWNHILL, John Richard, of Altrincham, Cheshire.
CASTLE, Charles Edward, of Orpington, Kent.
CHAN, Siu Lau, of Handsworth, Birmingham.
CHOUDHURY, Suhas, B.E., of London.
COOKE, Raymond Mary, B.E., of Galway, Ireland.
DAS GUPTA, Achintya Kumar, B.Tech.(Hons.), of London.
DHIR Om Parkash, B.A.(Hons.), of Punjab, India.
ENGINEER, Adi Jehangir, B.E., of Bombay, India.
GHOSE, Sushil, of Denton, Manchester.
GOPALAKRISHNA PILLAI, Ponnamma Sankarapillai, B.Sc., of Kerala, Southern India.
GRIFFIN, Joseph Richard Wyman, of Manchester.
HO CHEONG TUCK, of Kuala Lumpur, Malaya.
HUNT, Anthony James, of Wallingford, Berkshire.
KAMAT, Bhalchandra Achyutrao, B.E., of London.
KANHERE, Hari Balkrishna, B.E., of Bombay, India.
MARDELL, Denis Reginald, of Chessington, Surrey.
MUZUMDAR, Arun Madhukar, B.E., of Bombay, India.
NAGI REDDY, Kandula, B.E., of Warangal (A.P.), India.
ORANGUN, Cornelius Oluwasade, B.Sc.(Eng.), of Leeds.
PLAYLE, David Francis, of London.
READMAN, Peter, of Middlesbrough, Yorks.
SANYAL, Dipendra Kumar, B.E., of West Bromwich, Staffs.
SHAH, Bhikhalal Nahalchand, B.E., of Ahmedabad, India.
SIJUWADE, Adegboyega, of London.
SKELLAND, John, of Rainford, Nr. St. Helens, Lancs.
SOPHOCLIDES, Charalambos Michael, of Amersham, Bucks.
TAYLOR, Gordon Maurice, of Sutton Coldfield, Warwickshire.
TAYLOR, Robert Stewart, of Roade, Northamptonshire.
THOMSON, Hugh Melville, of Ash Vale, Surrey.
VENKATASUBRAMANIAN, Krishnier, B.Sc., of Calcutta, India.
WILSON, John Trevor, of Stockton-on-Tees, Co. Durham.

ASSOCIATE-MEMBERS

CHANDA, Kanak Kumar, B.Sc., of Oberengstringen/Zh, Switzerland.
CHAPMAN, Roy Dennis, of Middlesbrough, Yorks.
GORE, Prabhakar Narayan, B.E., of Maharashtra, India.
MACRAE, Herbert Tweddle, of Newcastle upon Tyne.
PERINI, Flavio, of London.

MEMBERS

HALL, John Charles, of Redcar, Yorks.
SATTLER, Konrad, Professor Dr.-Ing, Dr. tech.h.c., West Berlin, Germany.
TEATHER, Edgar, of Finham, Coventry.
WITHERICK, Reginald William, of Leeds.
WOODROW, Charles James, J.P., of Plymouth.

TRANSFERS

Students to Graduates

CLEVELAND, Richard Martin, of Beckenham, Kent.
PROBERT, Leslie Anthony, of Purley, Surrey.

Graduates to Associate-Members

CHATARJEA, Sukhendu, of Calcutta, India.
DAY, Thomas Michael, of Norwich.
DODD, James Michael, of Montreal, Canada.
FLEMING, Ian Doyle, B.Sc., of Natal, South Africa.
LEOW KHOON FONG, of London.
NIGHTINGALE, Peter, of Guelph, Ontario, Canada.
POOLE, James John, B.E., of Dublin.
PRABHU, Sudhakar Shrirang, B.Eng., of London.
REDDY, Dronnadula Venkatarama, B.E., M.S., Ph.D., of Visakhapatnam, India.
ZEEGEN, Alan Stanley, of London.

Associate-Members to Members

AU SIK-LING, of Hong Kong.
BROWN, Sydney Clarence, of Painswick, Glos.
FEATHER, Haydn, Milford, Nr. Stafford.
FORSBREY, Leonard William, of Loughton, Essex.
FOWKE, Bertram William, A.M.I.C.E., of Harrow, Middlesex.
HELAL, Joseph, B.Sc., of Beyrouth, Lebanon.
KAGANAS, Henechas, B.Sc., B.Eng., of Johannesburg, South Africa.
MOORE, Stanley George, of Solihull, Warwicks.
MOULD, George, B.Sc., A.M.I.C.E., of Orpington, Kent.
PITCHER, Edward Archibald, of London.
SANDERS, Reginald Newis, of Northampton.
TUCKER, Duncas Charles, B.Sc., A.M.I.C.E., of Sutton, Surrey.
WHEATLAND, Robert John, of London.

Retired Associate to Associate

ROBSON, Robert David, A.R.I.B.A., (Ret.), of Newcastle upon Tyne.

Membership to Retired Membership

GRIGGS, Frank Lewis, of Chislehurst, Kent.

OBITUARY

The Council regret to announce the deaths of George Allen ROBINSON, William Joshua GRIGGS (Retired Members); Edwin Walter DORRINGTON, Charles Frederick SHARPLES, James STUART, Bernard Howard KNIGHT, Harry JACKSON (Members); Alfred Henry WAKEFIELD (Associate); Thomas McMaster COLVIN (Associate-Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignation of Charles James WOODROW, (Graduate).

RE-ADMISSION TO RETIRED MEMBERSHIP

The Council have agreed that William Stephen BENTON, former Member and Retired Member, be re-admitted to Retired Membership of the Institution.

HONOURS AND AWARDS

In offering their sincere congratulations to the following members on the distinctions recently conferred upon them, the Council are also expressing the good wishes of the Institution.

KNIGHT BACHELOR—Professor J. F. Baker, O.B.E. (Member).

ORDER OF THE BRITISH EMPIRE—C.B.E.
Mr. W. A. Fairhurst (Member).

ORDER OF THE BRITISH EMPIRE—M.B.E.
Mr. W. J. Purcell (Associate-Member).

INSTITUTION AWARDS

Institution Gold Medal

The Gold Medal of the Institution has been awarded to Professor Felix Candela in recognition of his distinguished services to structural engineering in connexion with the development of concrete shell structures.

Certificate of Commendation

A Certificate of Commendation has been awarded to Colonel A. R. Mais, O.B.E.(Mil.), E.R.D., T.D., D.L. (Member), in recognition of his services to the Institution in connexion with the establishment of a Readership in Structural Engineering at Southampton University.

Grants from the Award of Merit and Scholarship Fund

The following awards in recognition of high standard attained in the Institution's Examinations have been made for the Session 1959-60 :—

Graduateship Examination

Mr. P. S. Parikh—ten guineas.
Mr. W. L. Jones—ten guineas.

MESSAGE FROM THE PRESIDENT

The President wishes to thank all those members who so kindly sent him their good wishes for his tour of Africa and Malaya; he will be making a full report on his return.

He also wishes to take this opportunity of thanking the contributors to the Maitland Testimonial. The Testimonial is to be presented to Major Maitland at a dinner to be held at the Dorchester Hotel on 2nd November, a notice regarding which will appear in a later issue of the journal. It will be necessary to close the Testimonial account on the 30th September so it is hoped that any members who intend to subscribe will send their contributions to arrive before that date.

The President also thinks that members would like to know that Major Maitland is now recuperating from the major operation which he underwent on the 31st May.

REPRESENTATION

The Council have appointed the following Institution Representatives :—

Manchester and District Advisory Council for Further Education, Post Advanced Building and Civil Engineering Advisory Committee :
Mr. J. B. STORY.

Regional Advisory Council for Further Education in the South West, Northern Area Building Advisory Sub-Committee :
Mr. C. J. D. BOXALL (re-appointed)

BSI Structural Sections in Light Aluminium Alloys, Technical Committee NFE/21 :
Dr. M. S. G. CULLIMORE.

Sunderland Technical College, Advisory Committee on Building and Civil Engineering :
Mr. E. G. TURNER.

National Council for Technological Awards, Board of Studies for Engineering :
Dr. S. B. HAMILTON (re-appointed)

TECHNICAL OFFICER

The Institution of Structural Engineers require the services of a Technical Officer. Candidates should be between the ages of 30 and 50 years. They should hold a University degree in engineering science and should have some experience in the practical field of structural engineering; experience in research would be an advantage. They should also have an interest in engineering literature. The duties include responsibility to the Secretary for the work involved in the scientific activities of the Institution and the servicing of the scientific committees.

The post is permanent and pensionable and carries a commencing salary of not less than £1,500 per annum.

Applications with details of experience should be addressed to the Secretary, 11 Upper Belgrave Street, S.W.1., not later than 11th August. The envelope to be marked in the top left hand corner "Science."

ADDITIONS TO THE LIBRARY

AMERICAN SOCIETY OF CIVIL ENGINEERS. *Transactions*, Vol. 125, Part I, 1960., New York, 1960.

BENSON, C. S. *Graphic Statics*, London, 1961. Presented by Mr. J. W. Fortey.

BRITISH STANDARDS INSTITUTION. *British Standards Yearbook*, 1961, London.

BROWNING, C. D. *Building Economics and Cost Planning*, London, 1961. Presented by Mr. Derek Bond.

CEMENT & CONCRETE ASSOCIATION. *Technical Reports:— TRA/345 An investigation of the behaviour of composite concrete beams. I Composite rectangular beams; TRA/347 Tests on a one-sixth scale model of a hyperbolic paraboloid umbrella shell roof*, London, 1961.

D.S.I.R. NATIONAL BUILDING STUDIES RESEARCH PAPER No. 30. *The Durability of Reinforced Concrete in Sea Water*, London, 1960.

D.S.I.R. NATIONAL PHYSICAL LABORATORY NOTES ON APPLIED SCIENCE No 21. *Measurement of Load by Elastic Devices*, London, 1961.

FLEMING BROTHERS (Structural Engineers) Ltd. *Steelwork Section Book, 9th Ed.* Glasgow, 1961.

GIBSON, J. E. *The Design of Cylindrical Shell Roofs, 2nd Ed.*, London, 1961.

GLUECKAUF, E. Editor. *Atomic Energy Waste. Its Nature, Use and Disposal*, London, 1961. Presented by Mr. T. C. Waters.

HALPERIN, D. A. *Building with Steel*, Chicago & London, 1960. Presented by Mr. C. S. Benson.

INSTITUTION OF MECHANICAL ENGINEERS. *Proceedings*, 1959, Vol. 173, London, 1961.

IRON & STEEL INSTITUTE. *Sixth Report of the Corrosion Committee*, London, 1959. Presented by Mr. S. E. Golledge.

KAYLOR, H. *Prestressed Concrete Simply Explained*, London, 1961. Presented by the Author.

LABOVITCH, C., Editor. *Directory of Opportunities for Qualified Men*, London, 1961.

LA LONDE, W. S. Jr., & JANES, M. F., Editors. *Concrete Engineering Handbook*, New York & London, 1961 (Reference Library).

NADAI, A. *Theory of Flow and Fracture of Solids, 2nd Ed.*, New York & London, 1950.

OPPENHEIMER, S. P. *Erecting Structural Steel*, New York & London, 1960. Presented by the Library of Congress.

SZECHY, C. *Foundation Failures*, London, 1961.

TAYLOR, C. P. and TURNER, L. *Reinforced Concrete Chimneys, 2nd Edition* revised by L. Turner, London, 1960. Presented by Mr. Leslie Turner.

TAYLOR, D. W. *Fundamentals of Soil Mechanics*, New York & London, 1948. Presented by Mr. T. F. Moran.

TIMOSHENKO, S. and GERE, J. M. *Theory of Elastic Stability, 2nd Ed.*, New York & London, 1961.

U.S. DEPARTMENT OF COMMERCE, BUREAU OF PUBLIC ROADS. *Standard Plans for Highway Bridge Superstructures*, Washington, D.C., 1956 (Reference library).

U.S. DEPARTMENT OF COMMERCE, NBS MONOGRAPH 26. *Development of High-temperature Strain Gauges*, Washington, D.C., 1961.

WALLEY, F. and BATE, S. C. C. *A Guide to the B.S. Code of Practice for Prestressed Concrete*, No. 115 : 1959, London, 1961.

WORLD CONFERENCE ON PRESTRESSED CONCRETE 1957. *Proceedings*, San Francisco, California, 1957. Presented by the Library of Congress.

RECENT PUBLICATIONS OF INTEREST TO STRUCTURAL ENGINEERS

TIMBER DEVELOPMENT ASSOCIATION LTD.

The Resistance of Certain Timbers in Sea Defence Groynes to Shingle Abrasion. Test Record B/TR/4. by A. C. Oliver and R. P. Woods.

Sea Defence Groynes. Information Bulletin B/IB/1, by A. C. Oliver and H. Richardson. 3s. 6d.

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH

Working Stresses for Structural Laminated Timber Forest Products Research Special Report No. 15. H.M.S.O. 4s.

Building Research Digest No. 8. (2nd Series). Built-up Felt Roofs. (72-36-0-8). 6d.

Building Research Digest No. 9. (2nd Series) April, 1961. Dry-lined Interiors to Dwellings. (72-36-0-9). 6d.

Building Science Abstracts Vol. XXXIV, No 1, January, 1961. Abstracts Nos. 1 to 185. (72-4-61-1). 3s. 4d.

Heat Bibliography 1959. (49-446-0-59). 22s.

National Building Studies Research Paper No. 32. Forces Applied to the Floor by the Foot in Walking. 1. Walking on a Level Surface, by F. C. Harper, B.Sc., Ph.D., W. J. Warlow, B. L. Clarke, B.Sc., January, 1961. (47-551-32). 3s 10d.

Road Abstracts Vol. XXVII, No. 12., December, 1960. Abstracts Nos. 1236 to 1344. (72-25-60-12). 2s. 10d Vol. XXVIII No. 1., January, 1961. Abstracts Nos. 1-139. (72-25-61-1). 2s. 10d.

Road Research Technical Paper No. 50. An Analysis of the Cost of Road Improvement Schemes, by R. F. F. Dawson, B.A., F.S.S., January, 1961. (47-110-50). 2s. 1d.

HOUSE OF COMMONS PAPERS

164. *Iron and Steel Board*, Development in the Iron and Steel Industry Special Report, 1961. 7s. 6d.

STANDING COMMITTEE DEBATES

Hyde Park (Underground Parking) Bill. Standing Committee F. 1st and 2nd sittings. March 1st and 8th, 1961.

COMMAND PAPERS

1321. *Report on the 11th Session of the General Conference of the United Nations Educational, Scientific and Cultural Organisation.* Paris, November 14th to December 15th, 1960. 1s. 5d.

1308. *Technical Assistance*, Miscellaneous No. 1, 1961. Technical Assistance from the United Kingdom for Overseas Development. 2s. 4d.

MINISTRY OF EDUCATION

Technical Education in Wales. Report of the Central Advisory Council for Education (Wales) (Chairman A. B. Oldfield-Davies, C.B.E.). Addysg Dechnegol yng Nghymru. Adroddiad y Cyngor ar addysg (Cymru). (27-345). 7s. 8d.

FOOD AND AGRICULTURE ORGANISATION (FAO)

Timber Bulletin for Europe. Vol. XIII, No. 3. Production, Trade and Price Statistics. Jan-Sept., 1959 and 1960. Market Review to November 30th, 1960. 7s. 7d.

INTERNATIONAL ATOMIC ENERGY AGENCY

Codes for Reactor Computations. Proceedings of the Seminar on Codes for Reactor Computations, sponsored by the International Atomic Energy Agency and held at Vienna, April 25 to 29, 1960. 49s. 9d.

BRITISH ROAD FEDERATION

Government Road Programme, June, 1961, B.R.F., 26, Manchester Square, London, W.1.

NATIONAL PHYSICAL LABORATORY

Notes on Applied Science 15. Modern Computing Methods. 2nd Edition, Completely Revised. (48-120-16-61). 22s 1d. 21. Measurement of Load by Elastic Devices, by B. Swindells, M.A., B.Sc., A.M.I.Mech.E., and J. C. Evans, B.Sc., Ph.D., 1960. (48-120-21). 3s. 4d.

PERIODICALS

Building Research Station Digest (2nd Series). 6s.

Timber Bulletin for Europe. (Quarterly 7s. 9d.) with supplements, £1 16s.

Road Abstracts. £1 13s. (Monthly 2s. 8d.).

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The following Honorary Officers and members of the Committee have been elected for the Session 1961-62 :—

Chairman : J. B. Story ; *Vice-Chairman* : A. S. Sinclair ; *Immediate Past Chairman* : Prof. W. Merchant ; *Hon. Auditors* : K. Norrey and G. Jones ; *Hon. Secretaries* : W. S. Watts and M. D. Woods. *Ordinary Members of Committee* : S. Howell, C. S. Booth, E. Stone, D. M. Brotton, S. Neile, J. A. Wyatt, W. Bates, W. Fitton, R. Gray, H. Heaton, J. S. Watts, Prof. A. W. Hendry.

STUDENT AND GRADUATE REPRESENTATIVES

R. N. Price (*Grad.*) and R. J. Miller (*Stud.*).

Joint Hon. Secretaries : W. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs., and M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

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Hon. Secretary: S. M. Cooper, M.I.Struct.E., "Applegarth," 56, Hyperion Road, Stourton, Stourbridge, Worcs.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: H. T. Dodd, A.M.I.Struct.E., Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwicks.

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Hon. Secretary: M. S. G. Cullimore, B.Sc., Ph.D., A.M.I.Struct.E., Queen's Building, University Walk, Bristol, 8.

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Chairman: G. S. Laing; *Vice-Chairman*: H. M. Irving; *Second Vice-Chairman*: E. Heney; *Hon. Secretary*: E. B. Kretzschmar; *Hon. Assistant Secretary*: (to be appointed). *Ordinary Members of Committee*: K. F. Bird, E. L. Margo, G. C. Lillicrap, J. G. Hay, A. E. Tait, F. B. Thompson, P. St. C. Michau, F. C. Ramsey.

Hon. Secretary Cape Section: R. F. Norris.

Hon. Secretary Natal Section: E. G. Bennett.

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Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

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SECTION NOTICES

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Hon. Secretary: A. Donald, B.Sc.(Hons.), A.M.I.Struct.E., 122, Matipo Road, Te Atatu, Auckland, New Zealand.

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Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

NIGERIAN SECTION

Hon. Secretary: A. Brimer, M.I.Struct.E., Brimer, Andrews and Nachshen, Private Mail Bag, 2295, Lagos, Nigeria.

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The following Honorary Officers and members of the Committee have been elected for the Session 1961-62 :—

Chairman: B. J. Dixon. *Hon. Secretary*: P. J. Carroll. *Ordinary Members of Committee*: J. D. Metcalf, R. L. Smith, M. B. Gill, T. J. Powell, T. F. Russell, B. J. P. Le Cesne Byrne.

Hon. Secretary: P. J. Carroll, M.E., A.M.I.Struct.E., A.M.I.C.E., A.M.I.C.E.I., 9, Laburnum Road, Clonskea, Dublin, Ireland.

SINGAPORE AND FEDERATION OF MALAYA SECTION

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The following Honorary Officers and members of the Committee have been elected for the Session 1961-62 :—

Chairman: J. D. Norfolk; *Vice-Chairman*: M. R. Hawkins; *Hon. Secretary*: C. J. Woodrow; *Hon. Treasurer*: A. N. Perkins; *Hon. Auditors*: F. J. Powell and F. W. Potter. *Ordinary Members of Committee*: E. G. Cove, E. W. Howells, F. W. Potter, F. J. Powell, H. J. Scoles, J. A. Smith, W. C. Tyler, L. F. Vanstone, W. H. Lynch.

Hon. Secretary: C. J. Woodrow, J.P., M.I.Struct.E., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.